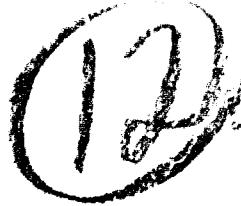


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TECHNICAL REPORT H-76-15

PROTOTYPE TESTS, OLD RIVER LOW-SILL CONTROL STRUCTURE, APRIL 1973-JUNE 1975

by

Frank M. Neilson, Allen R. Tool

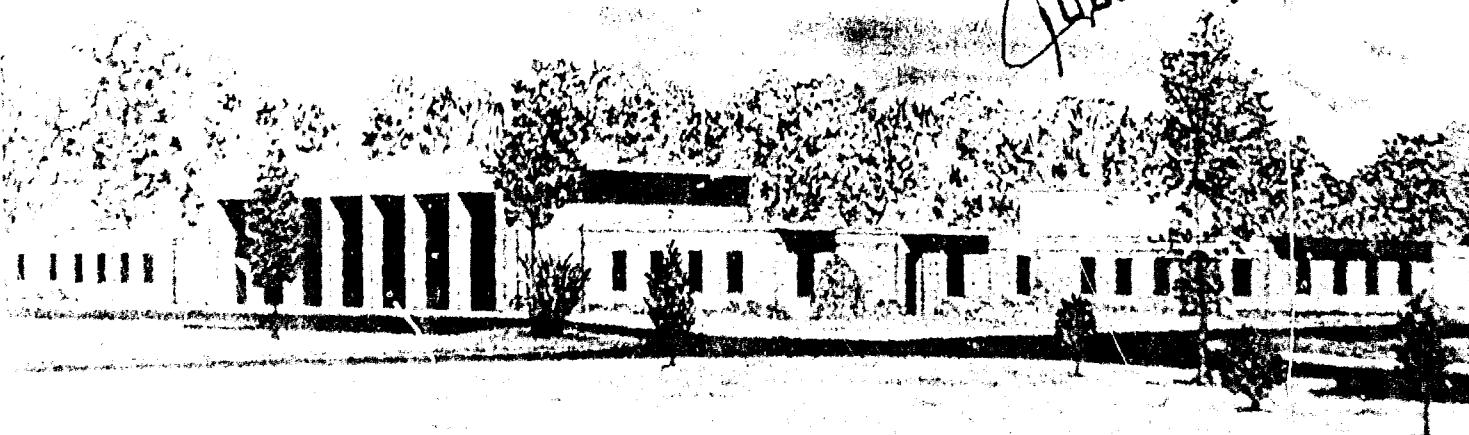
Hydraulics Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

September 1976

Final Report

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Prepared for U. S. Army Engineer District, New Orleans
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The following four separate series of prototype tests at Old River Low-Sill Control Structure are presented in this report. (1) High-flow vibration tests (4-5 April 1973), comprised of acceleration measurements, were made to determine the magnitude of the vibration of the structure under higher than normal flows. (2) Vibration monitor tests (16 April-1 June 1973), comprised of acceleration measurements, were made during a period in which the		(Continued)

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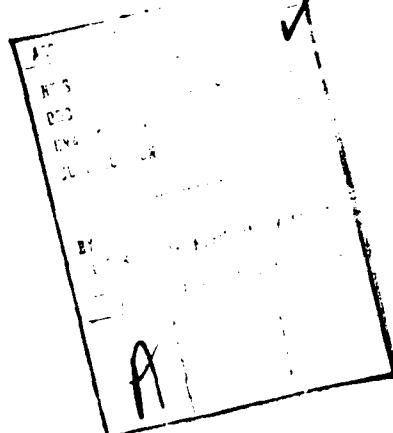
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20. ABSTRACT (Continued).

structure was not in a structurally sound condition. (3) Demolition tests (16-25 September 1973 and 4-15 November 1974), primarily acceleration measurements (with limited geophone and pressure cell data), were made during a period in which demolition activities (using explosive charges) were taking place near the structure. (4) Low-sill gate load tests (May-June 1975), comprised of measurements of strain, acceleration, load, and temperature, were made to determine the variations in loading that occur in the gate hoist mechanism and in the gate supports during both typical and nontypical gate operation.

Tests are briefly described and a limited sampling of the data is presented.



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PREFACE

The tests reported herein were performed under the sponsorship of the U. S. Army Engineer District, New Orleans. The tests were accomplished by the Prototype Branch, Hydraulic Analysis Division, of the Hydraulics Laboratory, U. S. Army Engineer Waterways Experiment Station (WES). Field instrumentation and recording, and electronic data reduction were accomplished by the Instrumentation Services Division of the WES under the general supervision of Messrs. L. M. Duke and J. L. Ferguson, respectively. Technical guidance during the demolition work was provided by Mr. Leo F. Ingram and Dr. S. A. Kiger of the Weapons Effects Laboratory at WES. Dr. F. M. Neilson and Mr. A. R. Tool prepared this report under the general supervision of Mr. E. D. Hart, Chief of the Prototype Branch, Mr. E. B. Pickett, Chief of the Hydraulic Analysis Division, and Mr. H. B. Simmons, Chief of the Hydraulics Laboratory.

Directors of the WES during the conduct of the tests and the preparation and publication of this report were BG E. D. Peixotto, CE, COL G. H. Hilt, CE, and COL John L. Cannon, CE. Mr. F. R. Brown was Technical Director.

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**CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT**

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
yards	0.9144	metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
kips (mass)	453.5924	kilograms
kips (force)	4448.222	newtons
pounds (force) per square inch	6894.757	pascals
inches per second	25.4	millimetres per second
feet per second	0.3048	metres per second
cubic feet per second	0.02831685	cubic metres per second
feet per second per second	0.3048	metres per second per second
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.



Figure 1. Old River Low-Sill Control Structure in operation. Flow is from Mississippi River (off top of photo) southwestward toward Red River. This photograph was taken before the 1973 flood.

PROTOTYPE TESTS, OLD RIVER LOW-SILL CONTROL STRUCTURE

APRIL 1973-JUNE 1975

PART I: INTRODUCTION

Scope and Purpose

1. This report is primarily a summary and chronology of a succession of prototype tests made to date on the Old River Low-Sill Control Structure (Figure 1). Eight test programs have been performed as follows.

<u>Purpose of Tests</u>	<u>Date</u>	<u>Results Reported</u>
1. High-flow vibration	2 Mar 1965 22-25 Apr 1965 9-11 Mar 1966	R. A. Yates ^{1,2,3,4,5}
2. High-flow vibration	4-5 Apr 1973	Telephone conversation, no formal report
3. Flood monitoring	16 Apr-Jun 1973	On-site, no formal report
4. Demolition monitoring	16-25 Sep 1973 4-15 Nov 1974	On-site, no formal report On-site, no formal report
5. Gate hoist loads	May-Jun 1975	Preliminary letter report ⁶

2. The concern in each of the first four sets of tests was structural vibration; however, the objectives of each program were distinctly different. During 1964, personnel at the project observed^{7,8} vibration of the structure at river stage 40 ft msl,* with a 5-ft head differential on the structure. Since these vibrations appeared to increase in intensity as the stage and head increased, a series of prototype tests during particularly high-flow conditions were undertaken (items 1 and 2 in the above tabulation). Typical pre-1973 results are presented elsewhere¹⁻⁵ and are not included herein. On 12 April 1973, during the

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

spring flood on the Mississippi River, four segments of the south upstream wing wall of the Low-Sill Control Structure collapsed. Collapse of the wall was caused by scour of the supporting foundation material. The initial failure was noted by project personnel; emergency repair work was immediately initiated by the U. S. Army Engineer District, New Orleans. A program to monitor the vibration of the structure (item 3 above) as an aid in anticipating additional structural problems for the duration of the emergency repair work was established shortly thereafter. Following the flood, the water level in the Mississippi decreased to such a level that demolition of the now detached wing-wall segments could be undertaken. The structural vibration was monitored (item 4 above) during the demolition work in order to help project personnel choose a satisfactory size of explosive.

3. Measurement of gate hoist loads (item 5 above) was undertaken during a period in which the existing adequacy of the much-repaired structure was being studied. The purpose of the hoist load tests was to help evaluate the feasibility of using the Low-Sill Control Structure gates for orifice flow control. Steady-state loading as well as vibration was measured.

Review of the Project

4. The Old River Low-Sill Control Structure is located on the western side of the Mississippi River about 312 river miles above Head of Passes. The structure is designed so that under project flood conditions 620,000 cfs are diverted from the Mississippi, through the Old River Low-Sill and Overbank Control Structure and outflow channel, into the Red River. The Red and Old Rivers combine to form the Atchafalaya as shown in the vicinity map in Figure 2. A view⁹ of the low-sill structure before the wing-wall failure during the 1973 flood is shown in Figure 1.

5. The Low-Sill Control Structure, constructed of reinforced concrete with vertical-lift gates,¹⁰ has a gross length of 566 ft between faces of abutment training walls; this length consists of eleven 44-ft gate bays, eight 8-ft piers, and two 9-ft central piers. The three central bays have a weir crest elevation of -5.0 ft msl for passing low

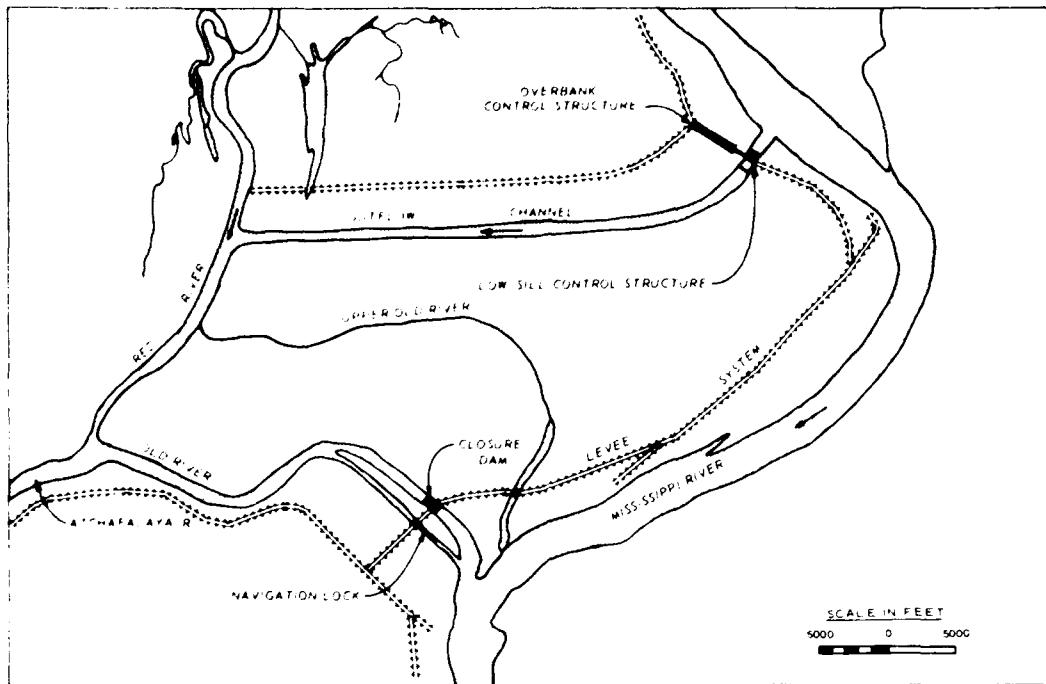


Figure 2. Vicinity map

flows. Four end bays on each side of the central portion have a weir crest elevation of 10.0 ft msl to permit the passage of substantially larger flows whenever the Mississippi River is above this elevation. Pertinent details and dimensions of the overall structure are shown in Plate 1. Typical piers are shown in Plate 2 and the south upstream wing wall (which collapsed during the 1973 flood) is shown in Plate 3.

6. Two general conditions that were of concern during design of the low-sill structure and that have limited the performance of the project since its construction are as follows:

- a. The structure is supported by an easily erodible, permeable, alluvial sediment. Preconstruction data are available regarding the geological¹¹ and physical¹² properties of the foundation materials. In addition, hydraulic model studies^{13,14b} provide information concerning areas of potential local erosion as well as information about flow patterns and sediment transport.
- b. The 11 gates in the structure are designed for flow regulation; the normal control is accomplished by the complete

closure of selected gate bays. Hydraulic model studies^{14a,14c} provide information regarding the operation and the hydraulic loading on the gates, as well as on the overall performance of the project as designed.

7. Since the project became operational in 1956, it has been noted⁸ that closure of a few gates, while others are held open, has resulted in large eddies (and local erosion) below the structure (see bottom of Figure 1). Since the 1973 flood, the structure has undergone substantial repair work. The project is currently being studied with regard to structural and operational modifications that will permit it to function according to design requirements.

PART II: HIGH-FLOW VIBRATION TESTS, 4-5 APRIL 1973

8. These vibration tests are the fifth series of similar field studies¹⁻⁵ concerning flow-induced vibration at the Old River Low-Sill Control Structure. The preceding series⁵ was during an unusually large differential head (Mississippi River stage 43.0 ft msl and outflow channel stage 13.5 ft msl) and indicated only small vibrations. For example, the calculated peak-to-peak displacements were less than 0.01 in. for all low-frequency (less than 5 Hz) Fourier series coefficients. An additional series of tests at a higher stage, rather than a larger differential head, was recommended⁵ which resulted in this series of vibration tests.

Instrumentation and Data Recording

9. Measurements were made at five locations on the structure as shown in Figure 3. Eight transducers (all accelerometers) were used. Transducers denoted by "L" are for horizontal measurements parallel to the direction of flow through the structure and normal to the axis of the structure (2L and 9La in Figure 3). The sign convention is that the measurement is positive whenever the structure is accelerated in the upstream direction. Transducers denoted by "T" are for horizontal measurements normal to the flow direction and parallel to the axis of the structure (3Ta, 17T, and 18T). The measurement is positive whenever the structure is accelerated in a northwesterly direction. Transducers denoted by "V" are for vertical measurements (1V, 7V, and 8Va); these measurements are positive for acceleration downward. The transducers were mounted on welded metal plates (four 5-in. sections similar to 48x6x1/2) which were clamped or epoxied in place. The transducers, mounting brackets, and clamps are shown in Figure 4.

10. The L and T transducers were unbonded strain-gage accelerometers with range of ± 0.25 g,* natural frequency of 15 Hz, and infinitesimal resolution; the damping is about 70 percent of critical. The V transducers were servo-accelerometers with range set to ± 0.2 g, natural

* g = acceleration due to gravity = 32.18 ft/sec².

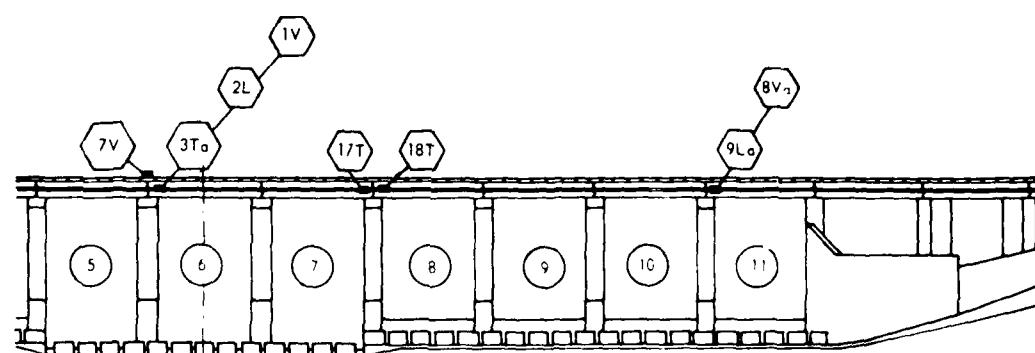
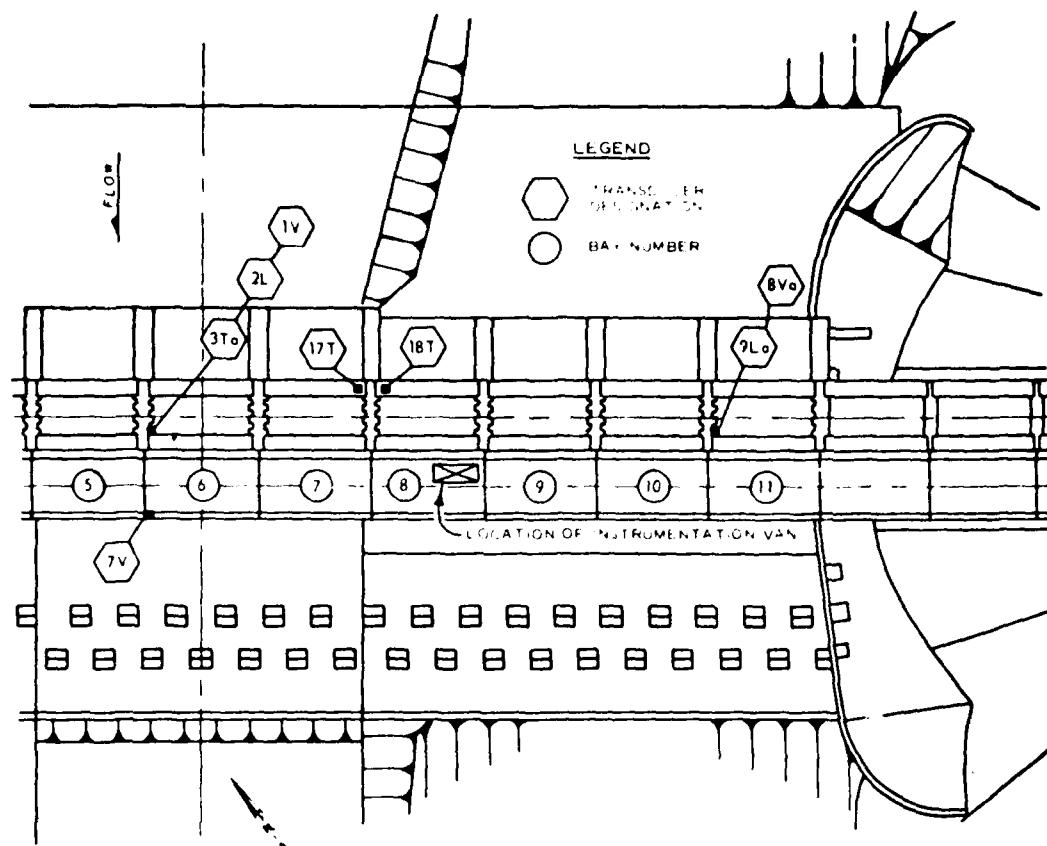
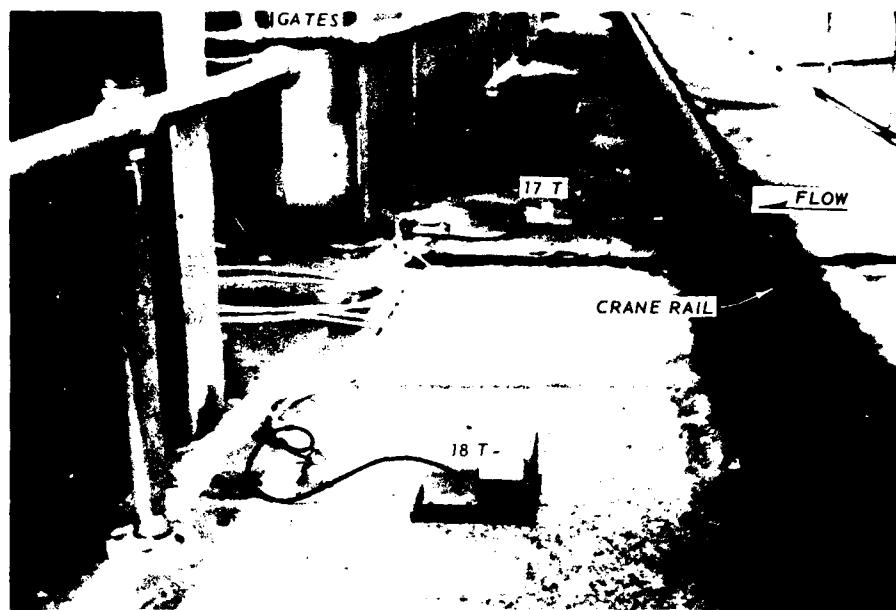
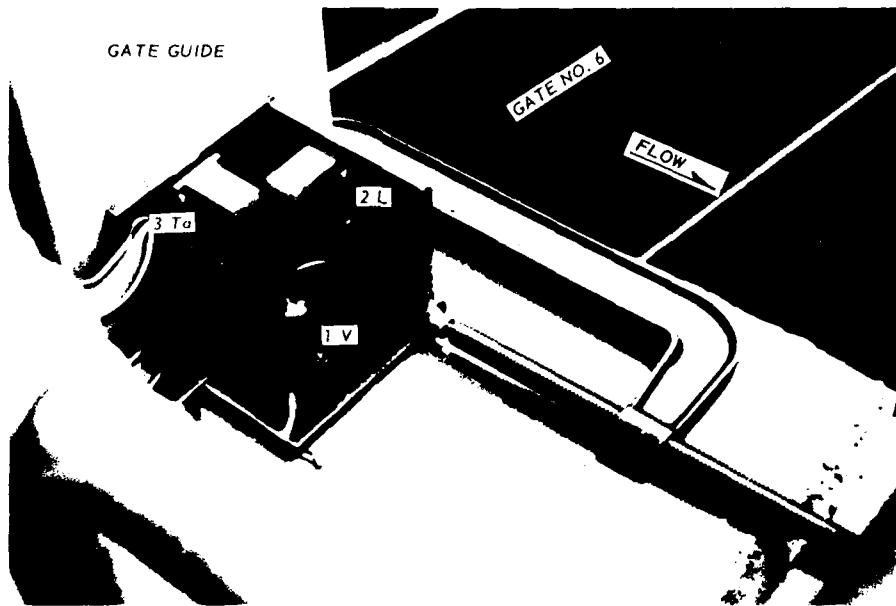


Figure 3. Transducer locations, 4-5 April 1973

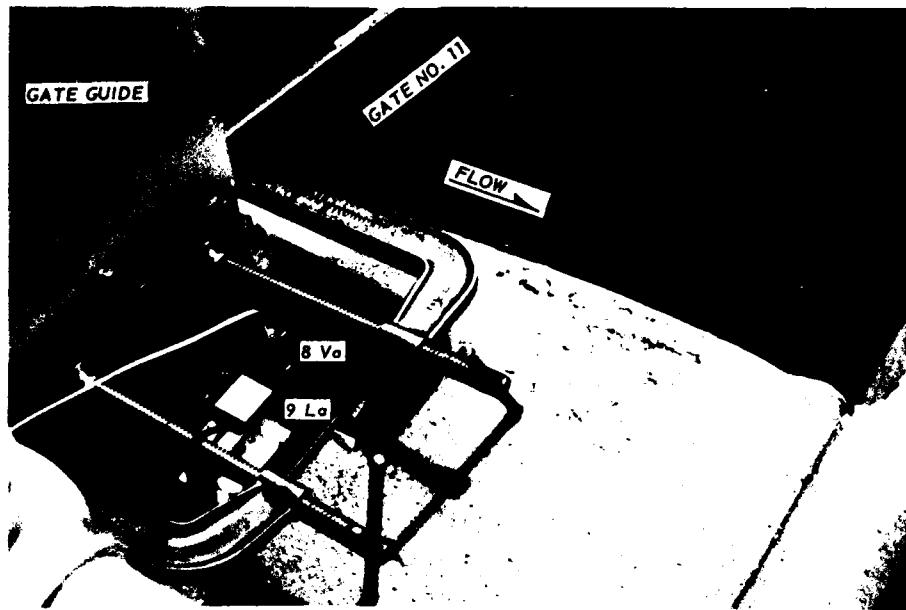


a. Transducers 17T and 18T on ends of bridge beams between bays 7 and 8 on the upstream side of the gates



b. Transducers 1V, 2L, and 3Ta on right, downstream gate guide of bay 6 for vibration of pier between bays 5 and 6

Figure 4. Transducers in place on structure,
4-5 April 1973 (sheet 1 of 2)



c. Transducers 8Va and 9La on right downstream gate guide of bay 11 for vibration of top of pier between bays 10 and 11



d. Transducer 7V on downstream handrail above the pier between bays 5 and 6 for vibration of cantilevered bridge support

Figure 4. (sheet 2 of 2)

frequency of 1000 Hz, and 5-micro-g resolution; the damping is about 60 percent of critical. The transducers were calibrated in the laboratory prior to installation in the field. As part of the calibration, the acceleration that results in a specific electrical signal at the recording device and a shunt resistance that causes the same signal level are obtained for each transducer. The shunt resistor is then used for calibration of the recording equipment in the field.

11. A schematic showing the main components of the data recording system is shown in Figure 5. The primary recording apparatus was the

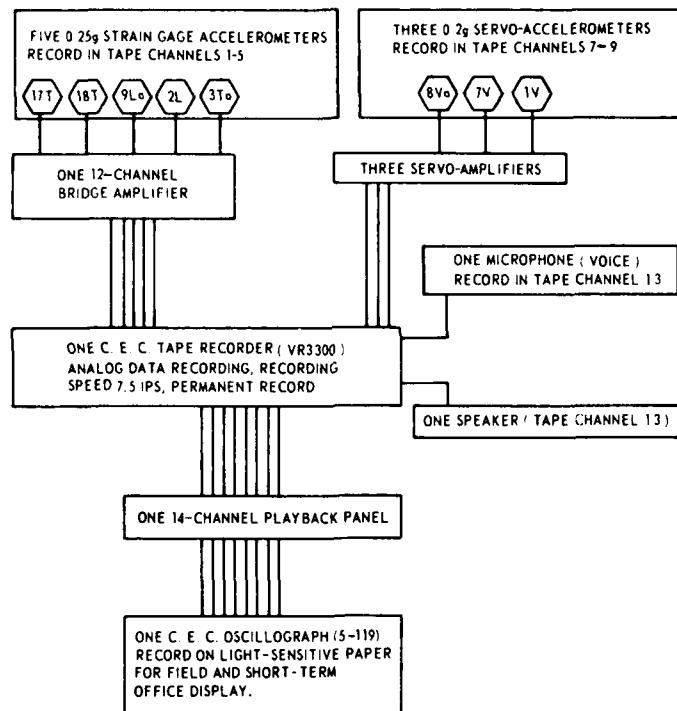
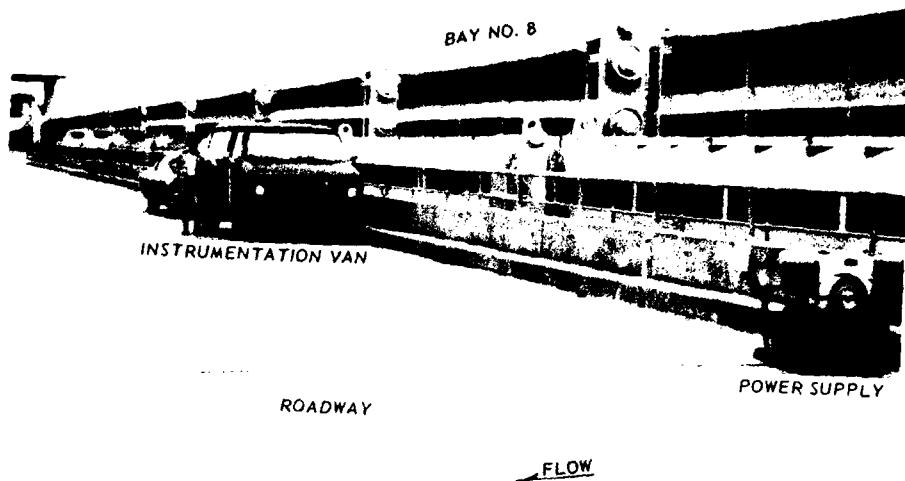


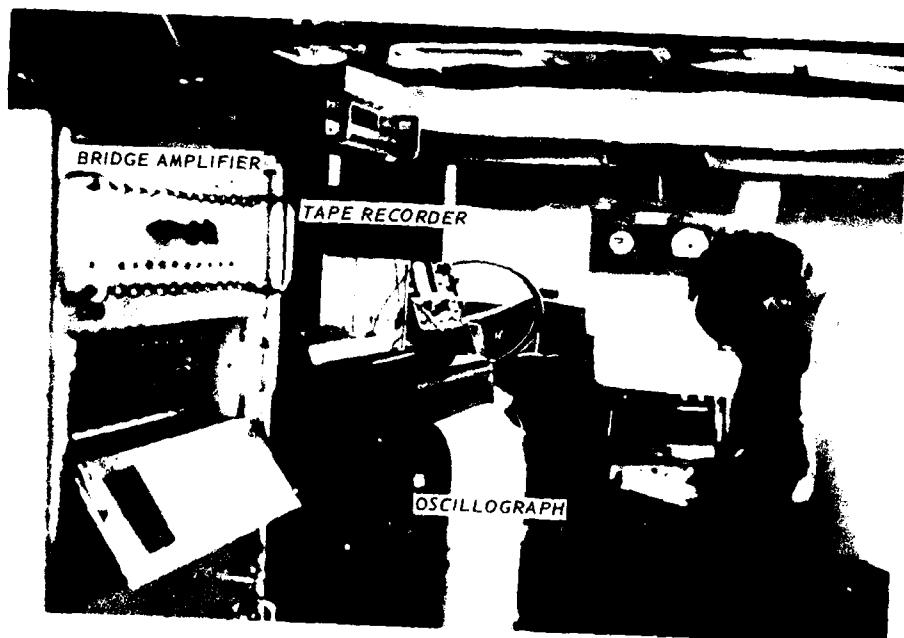
Figure 5. Schematic of data recording system,
4-5 April 1973

analog (FM) magnetic-tape recorder; an oscilloscope was used for short-term field and office display. Location of the instrumentation van and the 110-v power supply is shown in Figure 6a; recording equipment and interior of the van are shown in Figure 6b.

12. Only 9 of the 14 available magnetic tape data channels were



a. Van location on the bridge (looking northward)



b. Interior of the van

Figure 6. Recording equipment, 4-5 April 1973

used during the test program. The recordings were set up so that the shunt-resistor step calibration corresponds to a positive acceleration and to an increase in the FM magnetic-tape recorder frequency. The tape channels and calibration values are as follows:

<u>Channel</u>	<u>Transducer No.</u>	<u>Calibration Step, g's</u>	<u>Channel</u>	<u>Transducer</u>	<u>Calibration Step, g's</u>
1	17T	0.0096	8	7V	0.01
2	18T	0.0075	9	1V	0.01
3	9La	0.0077	10	Blank	--
4	2L	0.0095	11	Blank	--
5	3Ta	0.0085	12	Blank	--
6	Blank	--	13	Voice	None
7	8Va	0.01	14	Blank	--

Test Conditions and Field Observations

13. During the sequence of tests the 11 gates were fully open, the water-surface drop across the structure was about 4 ft, and the air and water temperatures were about 56°F; other conditions were as follows:

<u>Date 1973</u>	<u>Test No.</u>	<u>Test Period (hr:min)</u>	<u>Water-Surface Elevation ft msl</u>		<u>Total Flow* cfs</u>
			<u>Upstream</u>	<u>Downstream</u>	
4 April	1	17:15 to 17:25	56.0	52.0	445,000
5 April	2	10:47 to 10:57	56.5	52.4	450,000
	3	12:01 to 12:11			
	4	12:32 to 12:34			
	5	13:05 to 13:10			

* From model rating curves. ^{14c}

14. The primary calibrations for the five tests are electronic step calibrations prior to and following each day's tests. The L and T accelerometers were tilted prior to installation in order to verify the sign convention noted previously and to provide a rough check on the calibrations. During tests 1 and 4 above, there was considerable vehicle traffic on the bridge over the structure. During test 5, the microphone (voice channel) was directed downward from the bridge deck toward

the flow; the result was an uncalibrated recording of sound from the stilling basin.

15. Figures 7a and 7b indicate that undesirable flow conditions occurred along the south upstream wing wall; 7c shows that smooth flow occurred along the north upstream wall; and 7d shows the turbulent surface conditions that existed along the south downstream (stilling basin) guide wall. The extent to which the surface wave and eddy along the south upstream wall contributed to the subsequent failure of the wall cannot be determined from the subject test data. In fact, inspection of the data in the field indicated extremely small structural vibration, and none was felt when standing on the wall.

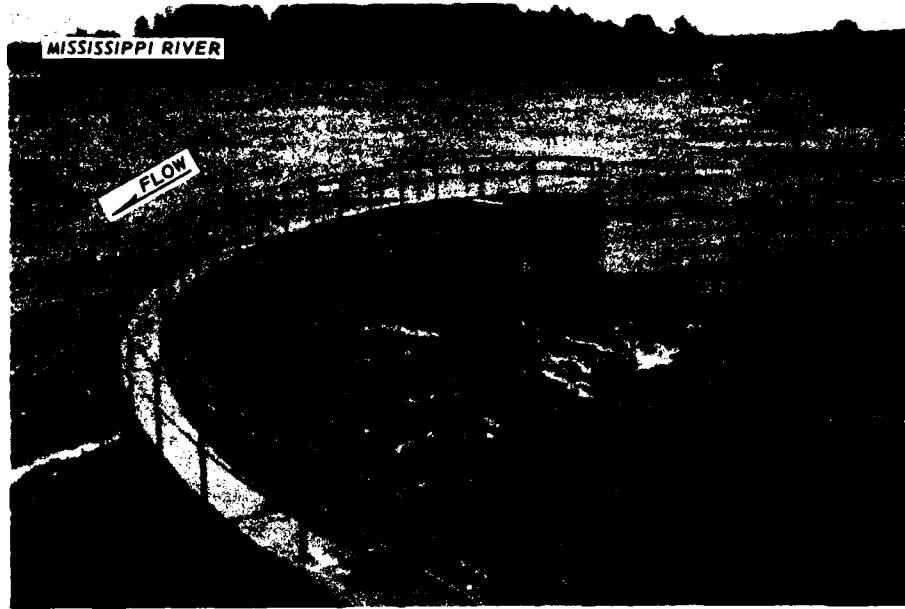
Data Reduction and Analysis

16. The primary interest in the measurements is low-frequency motion; for example, the previous tests⁵ were concerned with frequencies in the 1- to 5-Hz range. However, acceleration is highly frequency-dependent and the transducer response from low-amplitude, high-frequency motion may tend to mask the response from any low-frequency motion of interest. The frequency effect is present in the original field recordings and must be considered at each step in the data reduction as follows.

Original analog data

17. Instruments do not respond equally to simple sinusoidal motion at all frequencies; instead, higher frequency accelerations are damped¹⁵ as shown in Figure 8. For example, a transducer damped at 70 percent of critical indicates a measured acceleration equal to about 71 percent of the actual acceleration when the motion occurs at the natural frequency, f_n ,* of the transducer; at twice the natural frequency, $f/f_n = 2$, the response is only about 24 percent of the actual acceleration. The natural frequencies are not the same for all

* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

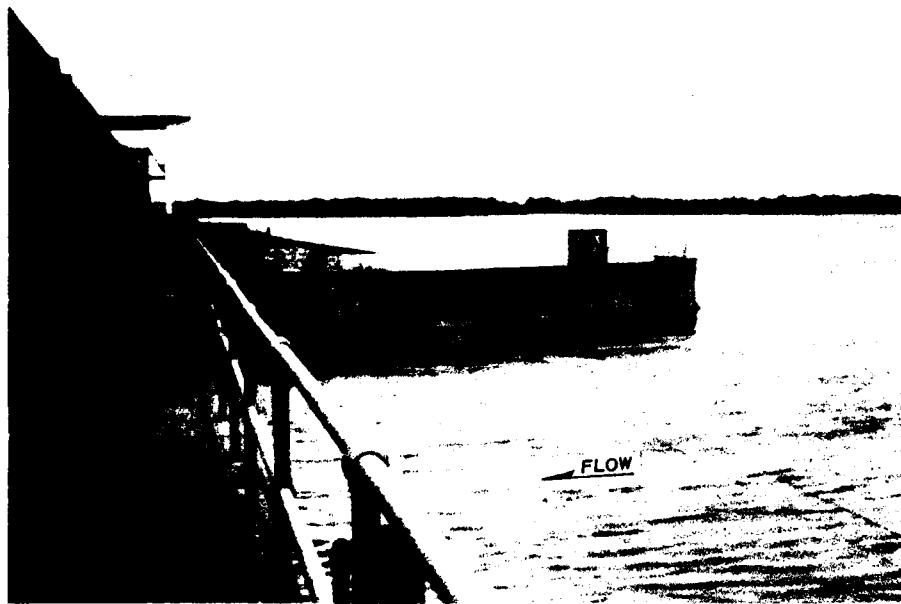


a. South upstream wing wall (note the large vortices in the lee of the wall)



b. South upstream wing wall (note large standing wave along the wall)

Figure 7. Flow conditions, 4-5 April 1973 (sheet 1 of 2)



c. North upstream wing wall



d. South downstream guide wall

Figure 7. (sheet 2 of 2)

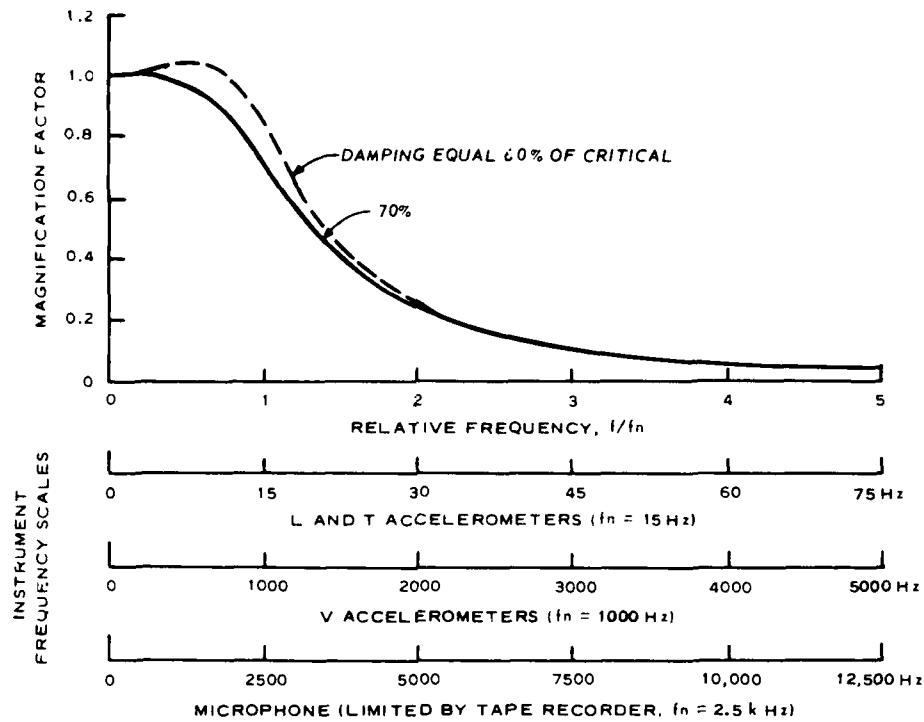


Figure 8. Frequency response, 4-5 April 1973

instruments; consequently, the L and T data are low frequencies ($f_n = 15 \text{ Hz}$); V data are intermediate frequencies ($f_n = 1000 \text{ Hz}$); and the audio records are high frequencies ($f_n = 2500 \text{ Hz}$).

Digitizing

18. Prior to analysis, the data are digitized; the consequence is that the data are "folded" at the Nyquist¹⁶ frequency, $f_N = 1/(2\Delta t)$, where Δt is the time (in seconds) between samples. This eliminates frequencies higher than f_N from direct consideration in subsequent analysis; however, some energy from significant higher frequency noise is retained in the digital values. The noise can be reduced by filtering either the analog or digital data; this is not done herein.* The L and T data are digitized at 102.4 samples per second and fold at

* The particular frequencies at which the energy is concentrated are of interest (rather than the amplitude of particular frequency components) and are apparent in the unfiltered data.

$f_N = 51.2$ Hz. Since f_N is substantially larger than f_n , folding is not significant for these data. On the other hand, the V and audio data are digitized at 1024 samples per second and fold at $f_N = 512$ Hz. Since the V and audio natural frequencies f_n are 1000 and 2500 Hz, respectively, and are considerably larger than f_N , these digital data contain significant high-frequency energy. Typical digital playbacks are shown in Plate 4 for test 5.

Frequency decomposition

19. The digitized time series data are transformed* into the frequency domain by means of a Fast Fourier Transform (FFT) computer program developed elsewhere.¹⁷ Transforms for the L and T data are shown in Plate 5 and for the V and audio data, in Plate 6. The value of the FFT here is simply that the dominant frequency components are immediately apparent. The main drawbacks to these FFT plots are: (a) although the phase** of each component is evaluated by the computer program, the resolution of the data into an equivalent acceleration at a single frequency remains a qualitative gesture of limited physical significance; and (b) without additional filtering, the possibility of the transferral of significant energy into the range of the FFT is a distinct possibility, particularly for the V and audio data as noted in the preceding paragraph.

20. The accelerometer data, test 5, are tabulated below with regard to maximum and minimum accelerations (a^+ and a^- , respectively) and dominant frequency, f_D . An equivalent simple harmonic peak-to-peak displacement, X , is calculated in the form

$$X = \frac{|a^+| + |a^-|}{(2\pi f_D)^2} \quad (1)$$

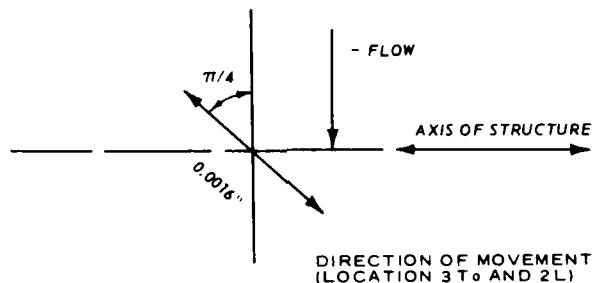
* The digitizing is performed electronically and the frequency decomposition via an EAI 640 digital computer by Mr. Cary Cox, Instrumentation Services Division, U. S. Army Engineer Waterways Experiment Station (WES). The basic elements of the Fast Fourier Transform computer technique used originate¹⁷ with the National Research Council, Ottawa, Canada.

** The plots of phase versus frequency are not included herein.

As indicated in the following tabulation, the displacements are small (all are less than 0.0015 in.), and the high-frequency motion is of particularly small amplitude (approximately 0.000003 in.).

Transducer	Acceleration, g's		Dominant Frequency Hz	Equivalent Peak-to-Peak Displacement, in.
	Maximum	Minimum		
2L	+0.00072	-0.00067	3.4	0.00118
9La	+0.00082	-0.00079	3.4	0.00136
3Ta	+0.00065	-0.00064	3.4	0.00109
17T	+0.00082	-0.00091	3.4	0.00146
18T	+0.00063	-0.00068	3.4	0.00111
1V	+0.00681	-0.00623	428.0	0.00000279
7V	+0.00827	-0.00722	--	--
8Va	+0.01677	-0.01349	299.0	0.00000331

21. The low-frequency components in the 2L and 3Ta data are roughly in phase (Plate 4) and they are nearly equal in amplitude (above). Therefore, the direction and the extreme peak-to-peak amplitude of the movement of the structure in a horizontal plane at this location are approximately as shown in the following sketch.



22. The audio FFT (Plate 6) indicates a broad noise band; the extreme low-frequency peak is probably due to the microphone support line responding to the movement of the structure and the spikes at 60 Hz and 120 Hz are probably due to electrical noise in the microphone amplifier.

Comparison with Previous Tests

23. The magnitudes of the measured transverse accelerations are approximately equal to those measured during 1966; the longitudinal

accelerations are substantially less than those observed in the March 1966 tests.⁵ For example, comparable peak-to-peak accelerations are as follows:

<u>Transducer</u>	<u>Peak-to-Peak Acceleration, g's</u>	
	<u>March 1966⁵</u>	<u>5 April 1973</u>
2L	0.0070	0.0014
9La	0.0041	0.0016
3Ta	0.0010	0.0013
17T	0.0012	0.0017
18T	0.0014	0.0013

The vertical accelerations are not directly comparable with those previously measured because of the large difference in accelerometer response characteristics. The marked reduction in longitudinal acceleration between the March 1966 tests and the April 1973 tests indicates that a large differential head (1966) causes more severe vibration than the high-stage condition (1973).

PART III: VIBRATION MONITORING, 16 APRIL-1 JUNE 1973

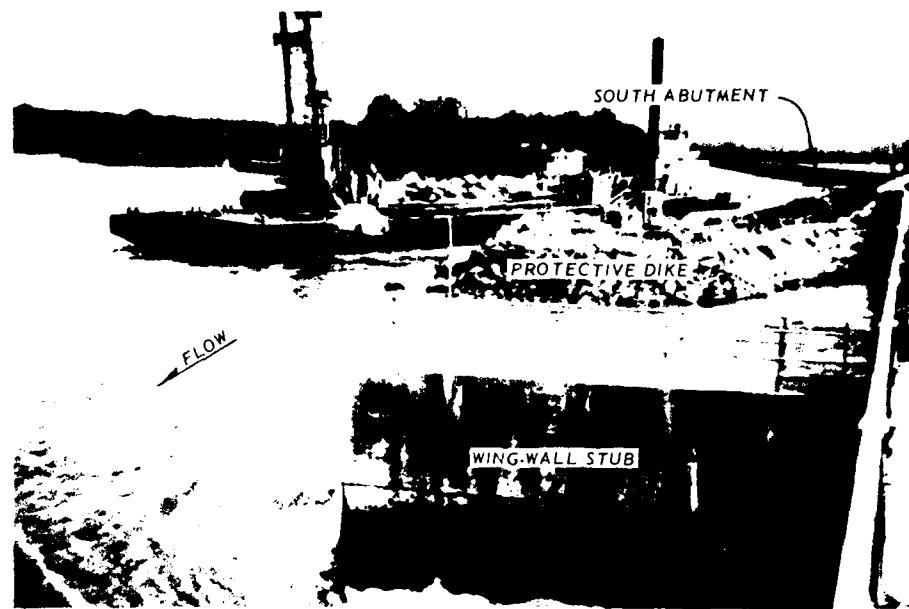
24. During 12-16 April 1973, the south upstream wing-wall segments A, B, C, D, and E in Plate 3 collapsed into the flow and were submerged upstream from the structure; emergency repair work was immediately initiated to protect the south abutment. The construction of a stone dike originating along the roadway south of the structure and proceeding into the flow above the remaining wing-wall stub was begun under Lower Mississippi Valley Division and New Orleans District direction. The extent of the dike and the wing-wall stub on 19 April are shown in Figure 9a. Additional structural deterioration was of concern and the vibration-monitoring program was initiated on 16 April as an aid in anticipating any additional major structural failure that might occur.

Instrumentation and Recording

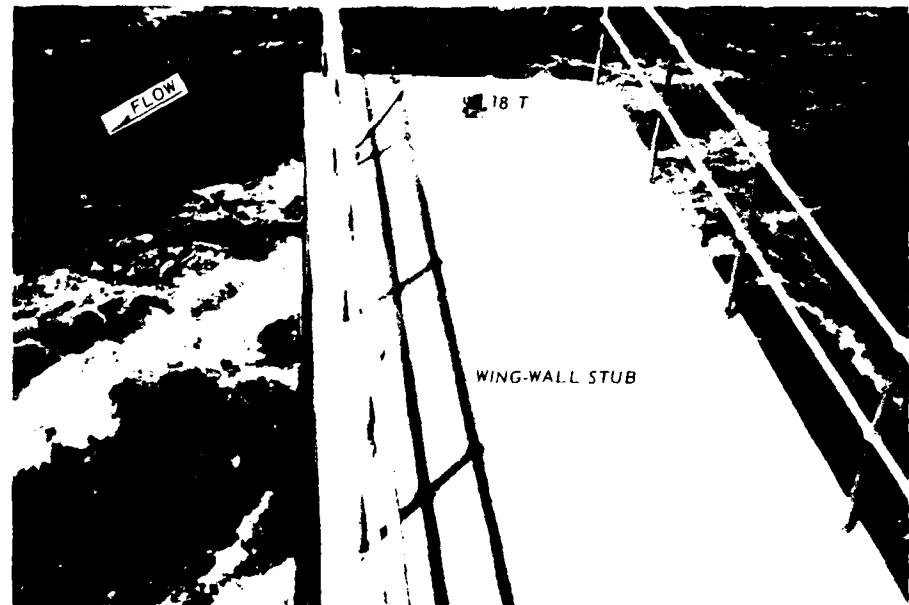
25. The measuring scheme set up during the high-flow vibration tests and described previously (Figures 4-6) was modified to accommodate the monitor program. Specific changes are as follows:

- a. All transducers were moved to new locations nearer to the construction area as shown in Figure 10. The new location (18T) on the stub, was of particularly acute interest (Figure 9b).
- b. Data from channel 18T were directly and continuously monitored via an oscilloscope; initially, all channels were recorded simultaneously for 5-min periods every half hour then reduced to every 3 hr (i.e., there are 8 complete tests for each day of monitoring at 3-hr intervals). The tapes were played back to an oscillograph following each test; these data were used for measurements and comparisons in the field. Forty-one 1-hr reels of magnetic tape were used during the complete monitoring program.

26. The following listing shows the channel location of the data on the magnetic tape and the calibration values on 16 April; subsequent exceptions to this listing occurred only whenever a particular data channel (transducer) became inoperable and was replaced or repaired in



a. Construction in progress on protective dike



b. Transducer 18T; the mount is epoxied to the top of the upstream end of the wing-wall stub

Figure 9. Wing-wall stub, 19 April 1973

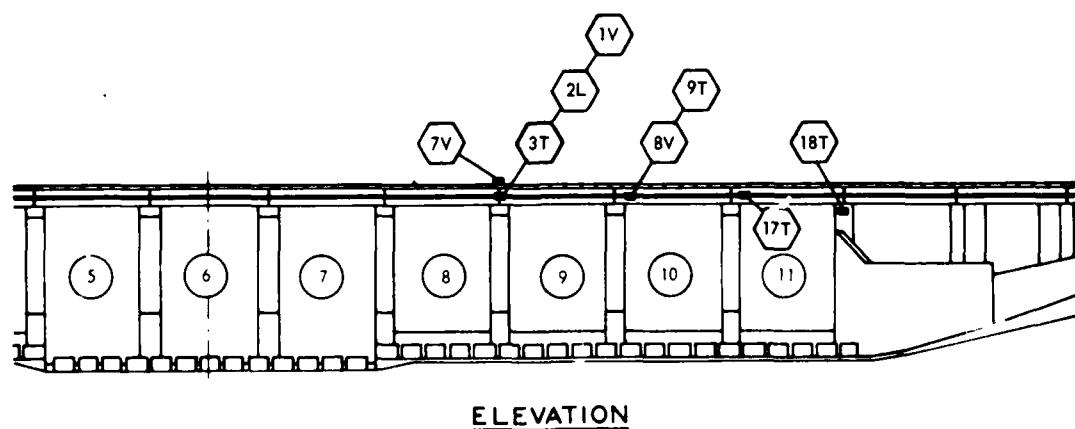
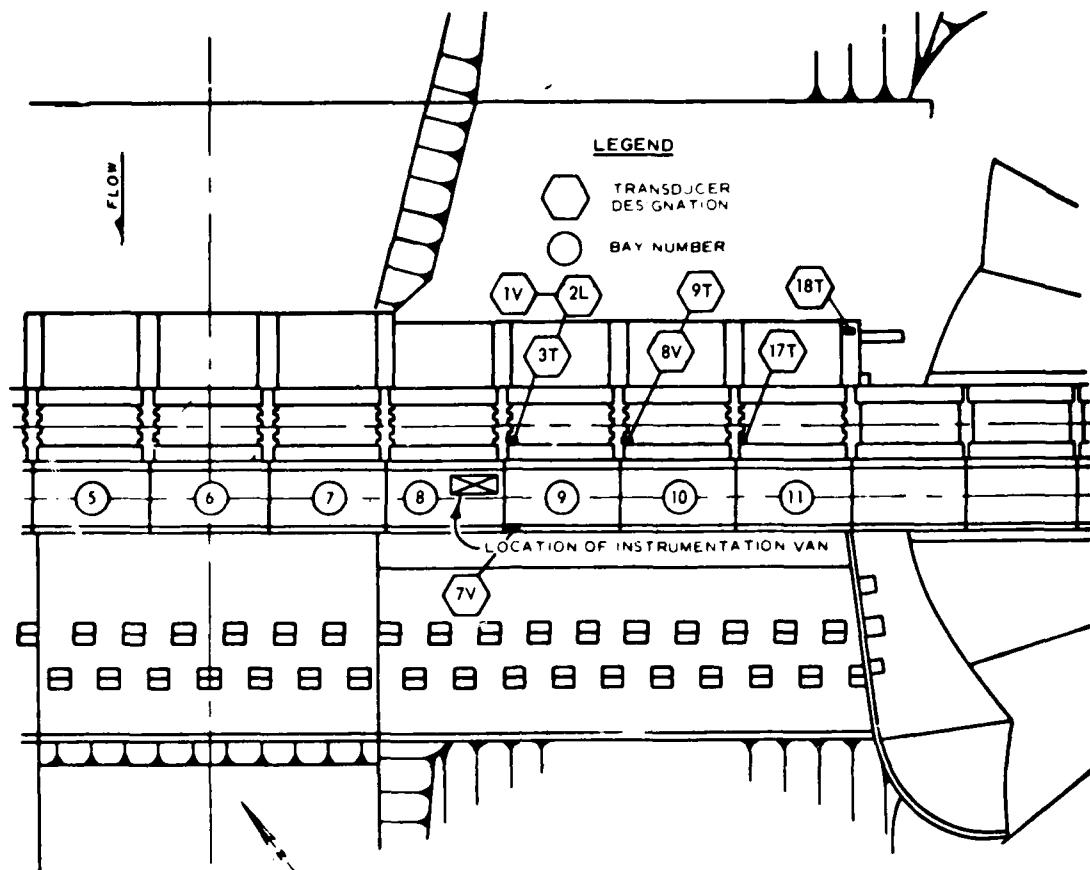


Figure 10. Transducer locations, 16 April-1 June 1973

the field. These occasions, not listed herein, are noted on the tape voice channel. The three transducers in brackets (9La, 3Ta, and 8Va) were moved to new locations for the high-flow vibration tests and were renamed 9T, 3T, and 8V; note that five new locations are denoted by symbols used in the 4-5 April tests and should not be identified as being at the same location.

Magnetic Tape Channel	Transducer No.	Range g's	16 April Calibration Step, g's
1	17T	0.25	0.0191
2	18T	0.25	0.0148
3	9T(9La)	0.25	0.0153
4	2L	0.25	0.0187
5	3T(3Ta)	0.25	0.0168
6	8V(8Va)	0.2	0.1000
7	7V	0.2	0.1000
8	IV	0.2	0.1000
A	Voice	--	None

27. The sign conventions regarding positive or negative accelerations and the significance of L for longitudinal, T for transverse, and V for vertical directions are as used previously (see paragraph 9).

Test Conditions

28. The hydraulic forces acting on the structure and the resulting structural response were of major concern during the vibration monitoring. Conditions which caused these items to change during the program are as follows:

- a. Natural changes in the stage of the Mississippi River and of the Old River outflow channel--these stages control the rate of flow through the structure and are shown in Figure 11. The large turbulent "boil" shown in Figure 12 occurred periodically* upstream from bay 10, apparently due to vortices shedding from the submerged wing-wall segments.

* An observer, using a stopwatch, noted that the period of time between occurrences of the large eddy ranged from about 5 to 70 sec on 16 April; the average period, during a 5-min observation time, was about 35 sec.

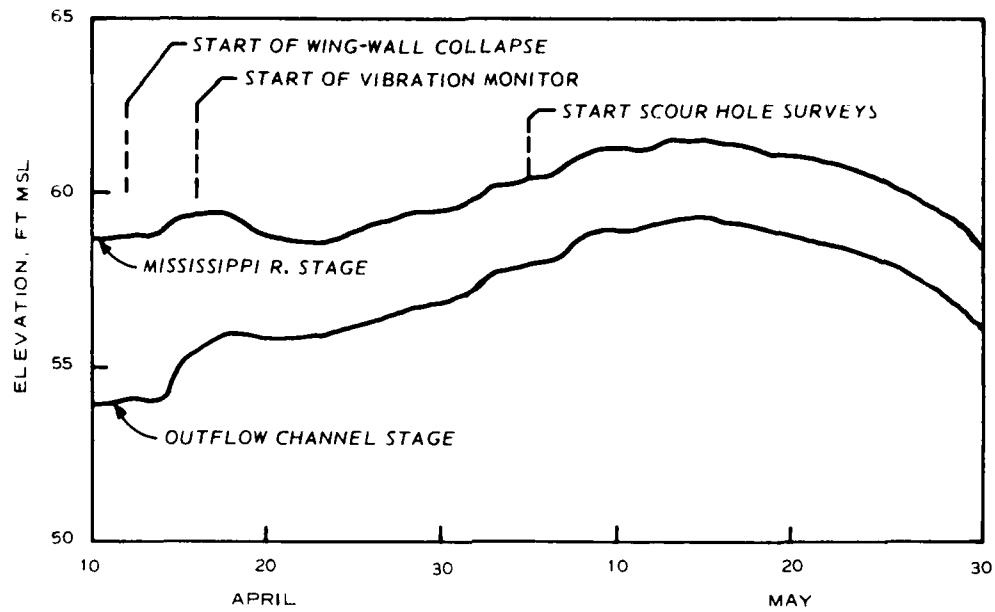


Figure 11. Hydraulic conditions during vibration monitoring (1973)



Figure 12. "Boil" in front of bay 10, 19 April 1973

- b. Extension of the protective dike above the wing-wall stub--this structure, which affects the flow pattern along the south abutment, is shown under construction in Figure 9a. The progression of the dike is illustrated in Figure 13; the outline shown for 4 May existed with only minor changes for the duration of the monitoring program.
- c. Erosion under the leading edge of the base slab--during surveys on 5 May a large scour hole was detected immediately upstream* from the structure; the results of a preliminary survey of the depression are shown in Figure 14. During the remainder of the monitoring program, derrick stone was being dumped upstream from the structure in order to terminate the erosion and to fill the scour hole. These hydrographic surveys of the depression were performed by New Orleans District personnel. The echo sounder transponder and receiver were suspended into the flow from a derrick attached to the gantry crane as shown in Figure 15.

29. Three other types of forces often caused increased vibration to occur. Vehicular traffic on the bridge deck, objects such as dumped

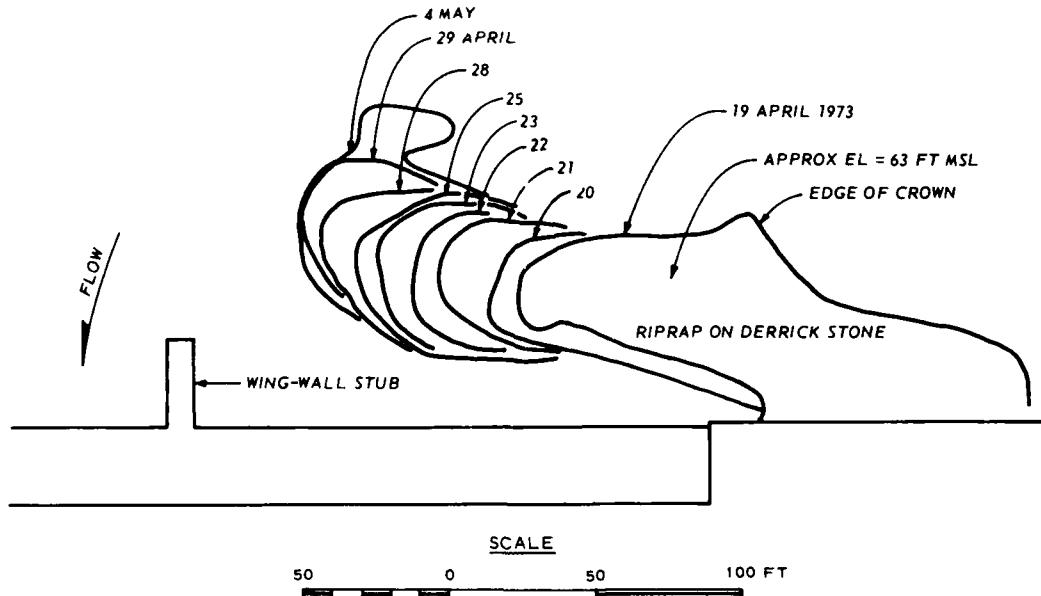


Figure 13. Progression of protective dike

* Subsequent field observations showed that the scour hole actually extended a significant distance under the base slab.

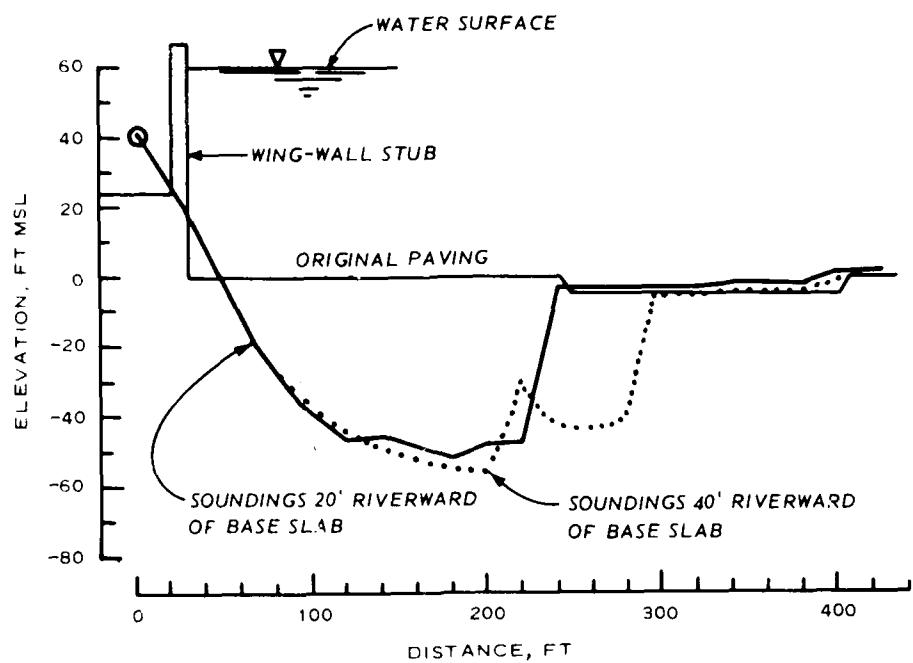


Figure 14. Scour hole geometry, 5 May 1973

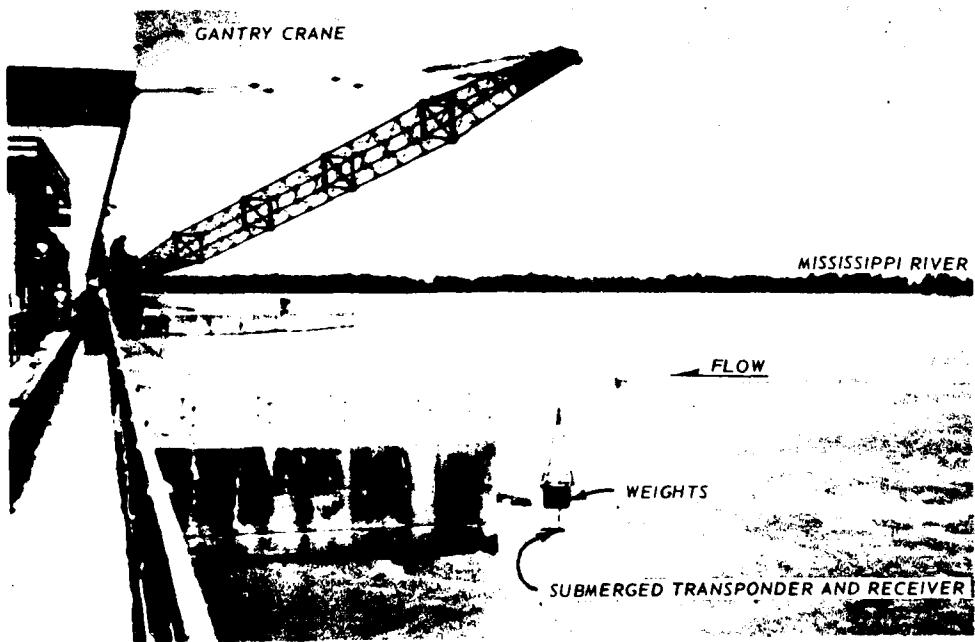


Figure 15. Surveying the scour hole, 5 May 1973

derrick stone impacting on the structure, and miscellaneous construction activity (particularly the movement of heavy equipment along the protective dike) resulted in short-term, higher frequency vibration. Although the accelerations during these occurrences were monitored and recorded at various times during the program, these data are not presented herein; the reason for this exclusion is that this high-frequency vibration is not a reliable measure of significant changes in the rigidity of the overall structure.

Data Reduction and Analysis

30. Three sets of 10-sec digital playbacks of the L and T data are shown in Plates 7, 8, and 9. These data are digitized at the rate of 100 samples per second ($f_N = 50.0$ Hz) and are representative of conditions during the following tests.

- a. Test 1 at 23:25 hr, 16 April (Plate 7)--the first recording in the monitor program.
- b. Test 158 at 12:00 hr, 6 May (Plate 8)--one of the first tests following the completion of the basic protective dike and was made on the day that the scour hole was first surveyed.
- c. Test 339 at 12:00 hr, 31 May (Plate 9)--the final test of the monitoring program; the scour hole had been filled with derrick stone.

31. The maximum and minimum L and T accelerations are tabulated below.

Transducer Location	Maximum Acceleration, g's			Minimum Acceleration, g's		
	Test 1	Test 158	Test 339	Test 1	Test 158	Test 339
2L	0.00036	0.00029	0.00039	-0.00034	-0.00034	-0.00025
3T	0.00055	0.00031	0.00046	-0.00049	-0.00029	-0.00037
9T	0.00050	0.00032	0.00041	-0.00046	-0.00035	-0.00041
17T	0.00048	0.00036	0.00051	-0.00063	-0.00040	-0.00061
18T	0.00074	0.00023	0.00033	-0.00072	-0.00022	-0.00038

32. Similarly, three sets of FFT's (see paragraph 19) are shown in Plates 10, 11, and 12 for Tests 1, 158, and 339, respectively. The

dominant low-frequency peaks shown in Plates 10, 11, and 12 are listed below along with computed peak-to-peak displacements. As before, the displacements are computed using maximum and minimum accelerations, above, and the dominant low frequency obtained from the FFT plots in Equation 1.

Transducer Location	Dominant Low Frequency Hz			Equivalent Peak-to-Peak Displacement, in.		
	Test 1	Test 158	Test 339	Test 1	Test 158	Test 339
2L	11.8	2.5	11.0	0.000048	0.000970	0.000052
3T	11.8	4.9	11.5	0.000074	0.000245	0.000061
9T	9.9	11.4	11.2	0.000094	0.000052	0.000064
17T	11.8	11.2	11.2	0.000077	0.000058	0.000087
18T	12.1	22.6	20.0	0.000097	0.000097	0.000016

Discussion of Results and Comparison with Previous Tests

33. Decompositions of the vibration monitor data (Plates 10, 11, and 12) do not show well-defined low-frequency peaks such as existed during the high-flow tests (Plate 5). Instead, each FFT shows a broad band of frequencies containing numerous significant peaks. The implication is that throughout the vibration monitor the various structural elements (piers, bridge deck, etc.) moved independently rather than as a single mass. Bearing in mind that duplicates of data are not available for the high-flow vibration tests and the monitor tests, and that the changes in water level between the two programs and during the monitor program influence the vibration to an unknown extent, the following items are presented as possible, rather than conclusive, occurrences.

- a. The foundation of the structure was in its most weakened state during test 158. For example, the longitudinal data show a lower significant frequency (2.5 Hz) than in either the high-flow test (3.4 Hz) or in the other monitor tests (11.8 and 11.0 Hz in tests 1 and 339, respectively). The longitudinal peak-to-peak displacements during test 158 and the monitor tests are not significantly changed (about 0.001 in.)

- b. The structure appears as rigid as at the start (test 1), judging by the frequencies and displacements shown above. In other words, the derrick stone apparently strengthened the structure as well as effectively terminating the erosion and filling the scour hole.

PART IV: DEMOLITION TESTS (16-25 SEP 1973 and 4-15 NOV 1974)

34. Following the monitor program, the vibration measuring system was temporarily demobilized and most of the equipment was returned to the WES. At various times during the subsequent repair work, through 15 November 1974, the system was reactivated at the Old River site; measurements during this period were either (a) during construction activity for which a technical custodian, who was responsible for equipment maintenance, made routine checks on structural vibration, or (b) during demolition of the wing-wall segments for which structural vibration data were required in order to specify a satisfactory size for the explosive charge. The former measurements are not described herein although undertaken for a wide range of construction activities, including the extended period during 1973-74 in which the scour hole was being filled with grout and the period during the spring of 1974 when the Mississippi River was again in flood. Instead, the data during the two demolition activities are presented in some detail since these data provide usable information regarding the basic dynamic response of the Old River Low-Sill Control Structure.

Overview of the Demolition Activity

35. Location and orientation of the wing-wall segments are shown in Figure 16. Segments B and C were located near the terminal point of the protective dike about 120 ft from the end of the wing-wall stub; the objective for the 16-25 July 1973 demolition was to sever these segments at grade (that is, at el 18 ft msl in Figure 16). Segments D and E were located about 50 ft upstream from the base slab in front of the pier between bays 9 and 10; the objective of the 4-15 November 1974 demolition was also to sever the segments at grade (that is, at el -1 ft msl in Figure 16). Segment A had apparently fallen into the scour hole and was already below grade.

36. Eighteen separate recording periods, denoted B1-B18 herein, were required for segments B and C; 106 recordings, B19-B124, were

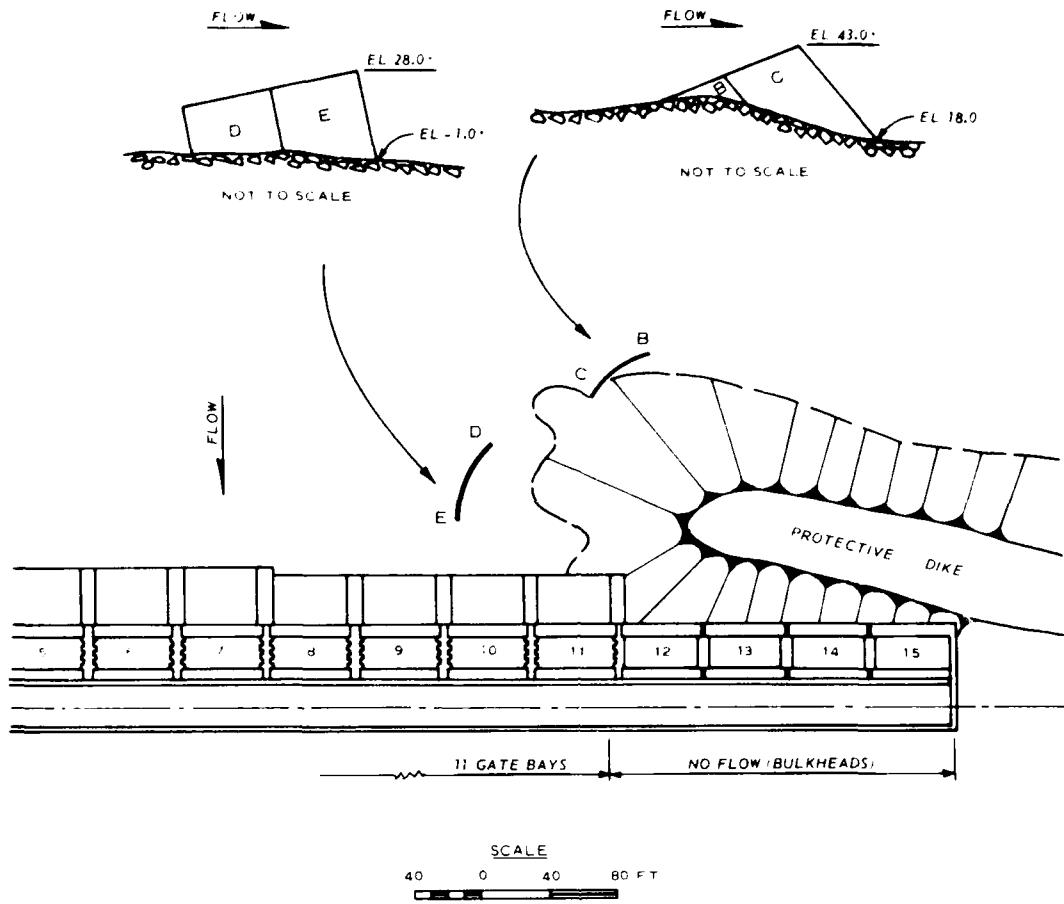


Figure 16. Location of the wing-wall segments

required for segments D and E. Tables 1 and 2 contain the sequence of events that make up the 1973 and 1974 demolition programs, respectively. Photographs showing the exposed portion of segment C, the contractor's personnel assembling the explosive charges, the water surface immediately after an explosion, and the final New Orleans District hydrographic survey are presented in Figures 17a, b, c, and d, respectively.

Instrumentation and Recording

37. The measuring scheme described previously (see Figures 4 and 5) was modified substantially to accommodate the demolition programs.



a. Wing-wall segments B and C,
9 October 1973



b. Molding point charges (line charges
in foreground), 21 October 1973



c. Immediately following blast B2,
9 October 1973



d. Damage survey in progress (NOD),
25 October 1973

Figure 17. Demolition activities, segments B and C

In addition, modifications regarding instruments and recorders used occurred at several instances during the program. The overall instrumentation is summarized in the following tabulation; specific features of the measuring scheme are described in the succeeding paragraphs.

Tape Channel	Custodian and		Tests		
	Test B1 Recorder 1	Test B2 Recorder 1	Tests B3-B18 Recorder 1	Recorder 2	B19-B124 Recorder 1
1	67L	67L	56L	P1	56L
2	67T	67T	56T	P2	56T
3	67V	67V	56V	P3	56V
4	78L	78L	78L	P4	78L
5	78T	78T	23L	Blank	78V
6	89L	89L	1112L	P5	89V
7	89T	89T	1112L2	13V1	90L
8	90L	90L	90L	13V2	90T
9	90T	90T	90T	13V3	90V
10	90V	90V	90V	P6	1011L
11	G7L	90V2	23V	Blank	1011V
12	G7T	1011V	78V	Blank	P1
13	G7V	67V2	1112V	Zero det	P2
14	Blank	Blank	IRIG	IRIG	IRIG
A	Voice	Voice	Voice	Voice	Voice

38. The following comments pertain to items mentioned in the above listing.

- a. Tape recorder 1 was operated at 7.5 in./sec for tests B1, B2, and B3-B18; 120 in./sec during tests B19-B48; and 60 in./sec during tests B49-B124. Recorder 2 was used only during tests B3-B18 and was operated at 120 in./sec.
- b. The accelerometer designations are changed from the previous tests. Specifically, the numerals preceding the L, T, or V designation were changed so that the transducer symbol is related to the point of attachment on the structure. For example, transducer 67L measured longitudinal acceleration and is located on the top of the pier between bays 6 and 7; 1112V measures vertical acceleration and is located on the south abutment.
- c. Accelerometers G7L, G7T, and G7V are strain-gage A69TC-5-350 accelerometers whose range equals ± 5 g's, natural frequency equals 375 Hz, damping equals 70 percent of critical, and resolution equals infinitesimal. These transducers were located on the fully open gate in bay 7

while the system was under the care of a custodian at the site, preceding the first demolition work.

d. Accelerometer mount changes were made during the program; the original mountings were similar to those during the previous program (Figure 4). The sequence of changes was as follows:

- (1) Following B1, accelerometers G7L, G7T, and G7V were placed under sandbags on the top of piers; the result was similar vertical measurements during B2 at two locations (67V2 was under a sandbag on the same pier as 67V, and 90V2 was under a sandbag near 90V) and a vertical measurement at one new location, 1011V.
- (2) During B3-B18, all vertical accelerometers were under sandbags rather than being clamped to the structure. The mounting bracket for transducer 1112L2 was clamped to a ladder on the south abutment pier at el 36 ft msl.
- (3) During B19-B124, the accelerometer mounting brackets were epoxied to the top surface of the piers.

e. The changes in the accelerometer locations between each of the tests are summarized in the following tabulation. Locations of the accelerometers during tests B10 and B60 are shown in Figures 18 and 19, respectively.

Tests	Accelerometer Type	Range +g's	Original Locations	
			Location Changes	
			Add	Delete
B1	Servo	2.5*	67V, 90V	
	Strain gage	0.25	67L, 67T, 78L, 78T, 89L, 89T, 90L, 90T	
	Strain gage	5.0	G7L, G7T, G7V	
B2 B3-B18	Strain gage	5.0	90V2, 1011V, 67V2	G7L, G7T, G7V
	Servo	2.5*	56V	67V
	Strain gage	0.25	56L, 56T, 23L, 1112L, 1112L2	67L, 67T, 78T 89T, 89L
	Strain gage	5.0	23V, 78V, 1112V	90V2, 1011V, 67V2
B19-B124	Servo	2.5*	78V, 1011V	23V
	Strain gage	0.25	1011L	23L, 1112L, 1112L2
	Strain gage	2.5	89V, 90V	--
	Strain gage	5.0	--	1112V, 78V, 23V

* The range of each servo-accelerometer is set by the user.

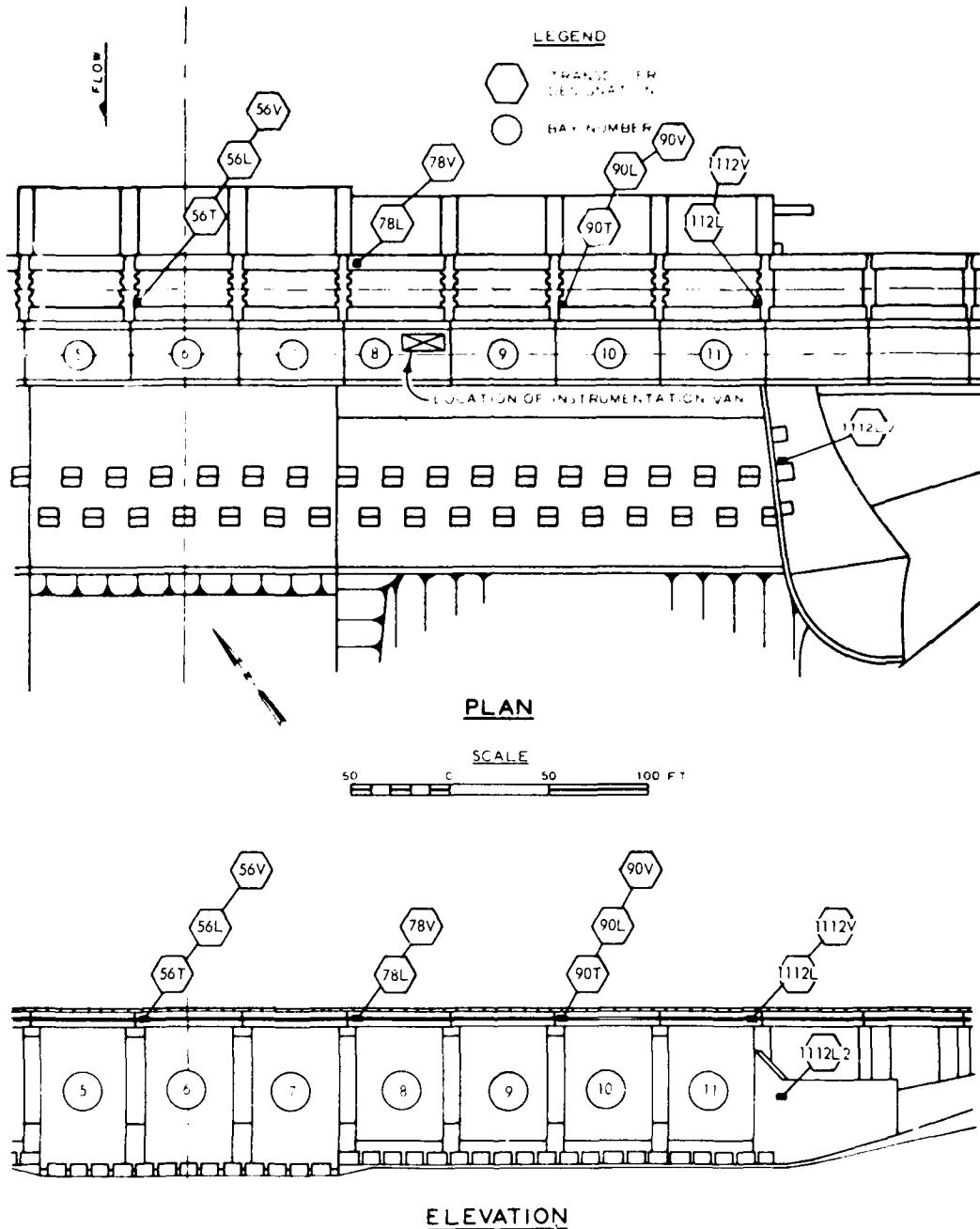


Figure 18. Accelerometer locations, test B10, 22 October 1973

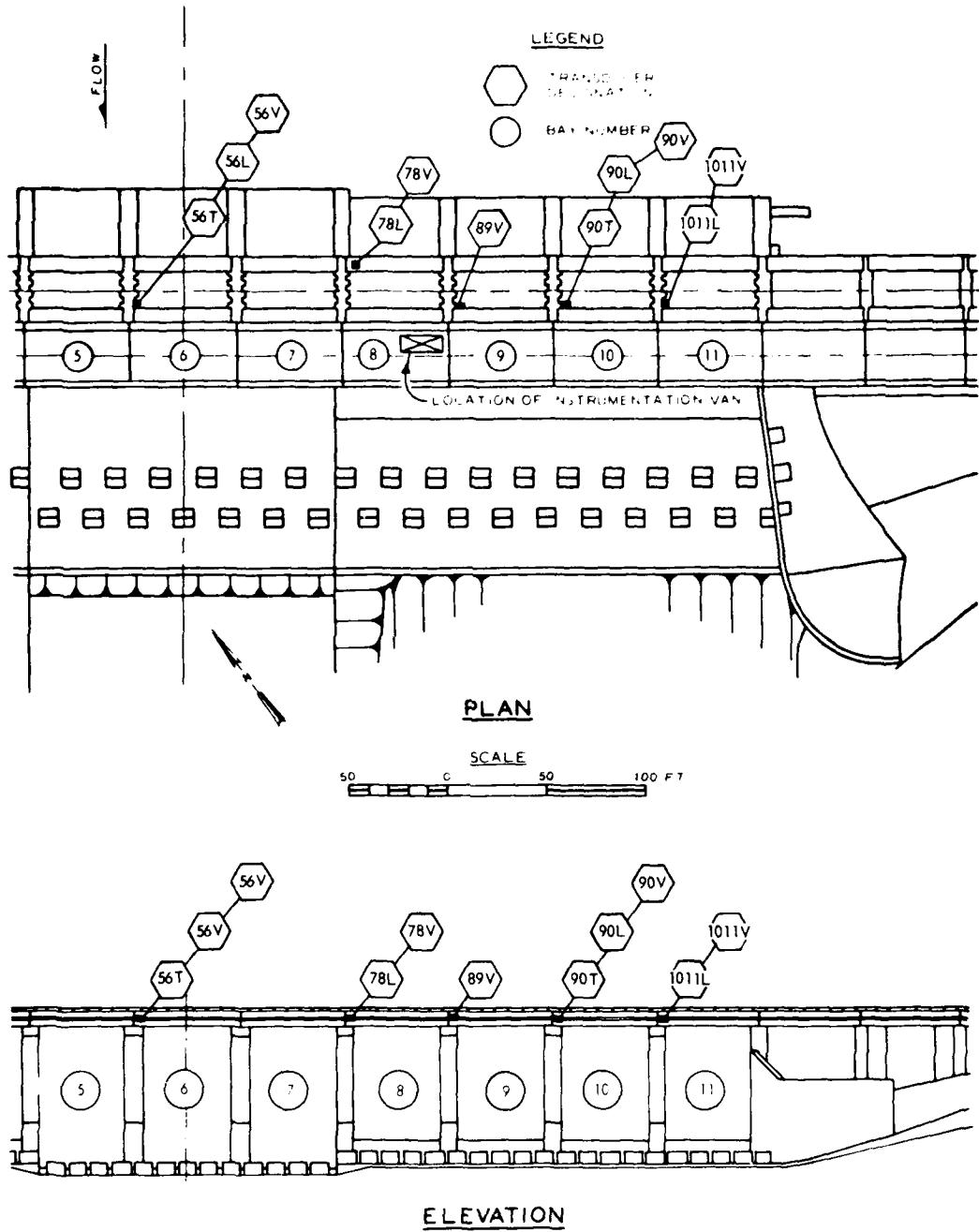


Figure 19. Accelerometer locations, test B60, 12 November 1974

f. Recordings other than acceleration are as given below:*

<u>Pertinent Tests</u>	<u>Items</u>	<u>Remarks</u>
B1-B124	Voice	An audio record of observations during the tests is retained on one channel of each tape record.
B3-B124	IRIG	This is the time code; during tests B3-B18 it is used to match occurrences between the two tape records.
B3-B18	P1,P2,P3,P4,P5 and P6	These are piezoelectric (high-frequency response) pressure transducers; the first four were suspended in 1-ft-diam pipes extending through the base slab and into the water-filled cavity beneath the structure. Transducer P5 was suspended by means of a wire rope attached to a crane hoist and was located about 10 ft below the water surface near wing-wall segments B and C; P6 was suspended into the flow near the leading edge of the wing-wall stub.
B5-B18	13V1,13V2,13V3	These are devices for measuring the vertical velocity of the foundation material of the structure; they were imbedded about 1 ft into the soil at locations indicated in Figure 20.
B3-B18	Zero det	Activation of the detonation device triggers a pulse to give a zero reference-time signal.

(Continued)

* Detailed descriptions of these transducers are omitted since these data are only of superficial interest herein.

<u>Pertinent Tests</u>	<u>Items</u>	<u>Remarks</u>
B19-B124	P1, P2	These are piezoelectric pressure cells; P1 was suspended from the midspan of bay number 10; P2 was suspended from the wing-wall stub.

Test Conditions

39. The river and outflow channel stages are presented in Figure 21a during tests B1-B18 and in Figure 21b during tests B19-B124. Other pertinent test conditions are as given previously in Tables 1 and 2. New Orleans District personnel periodically made surveys along the top of the structure; measured elevations at five points along the top of the structure are shown in Figure 22a over the period in which tests B1-B18 were made. These data indicate no significant trend of movement during the demolition of segments B and C. Prior to tests B19-B124, the foundation of the structure had been reinforced with a grouting compound and any settlement during these tests was also not of an amount measurable by standard survey technique.

Field Observations

40. A copy, via microfilm, of the oscillograph field record for test B10 is shown in Plates 13 (longitudinal and transverse accelerations) and 14 (vertical accelerations, vertical earth-motion velocities, and pressures). These records, and similar ones for each test, were used in the field for assessing the relative effects of the different amounts of explosive used. The following comments pertain to these records as they were viewed in the field.

- a. Accelerations. In interpreting the oscillograph charts, the extreme longitudinal and transverse accelerations were kept close to ± 0.1 g;* however, vertical accelerations

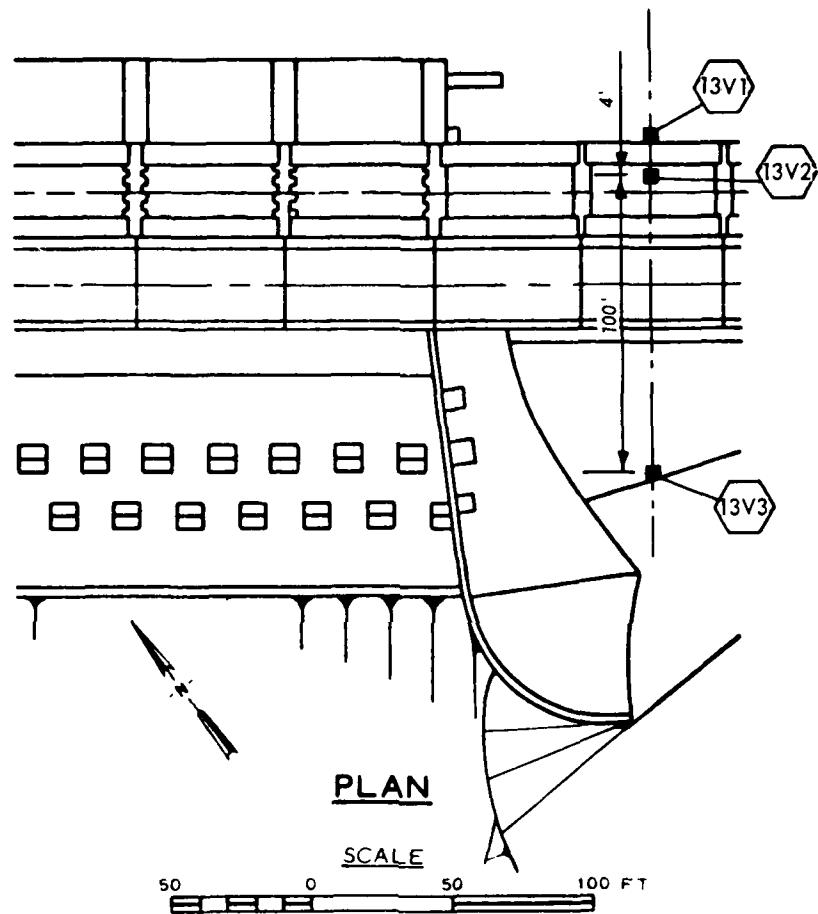
* For seismic shock problems¹⁵ a "caution" zone corresponds to an acceleration of 0.01 g, the limit for "probable" safety from damage is at 0.1 g, and the "definite" damage point is at 1.0 g.

LEGEND

HEXAGON TRANSDUCER DESIGNATION

CIRCLE BAY NUMBER

FLOW



ELEVATION

Figure 20. Location of geophones, Test B10, 22 October 1973

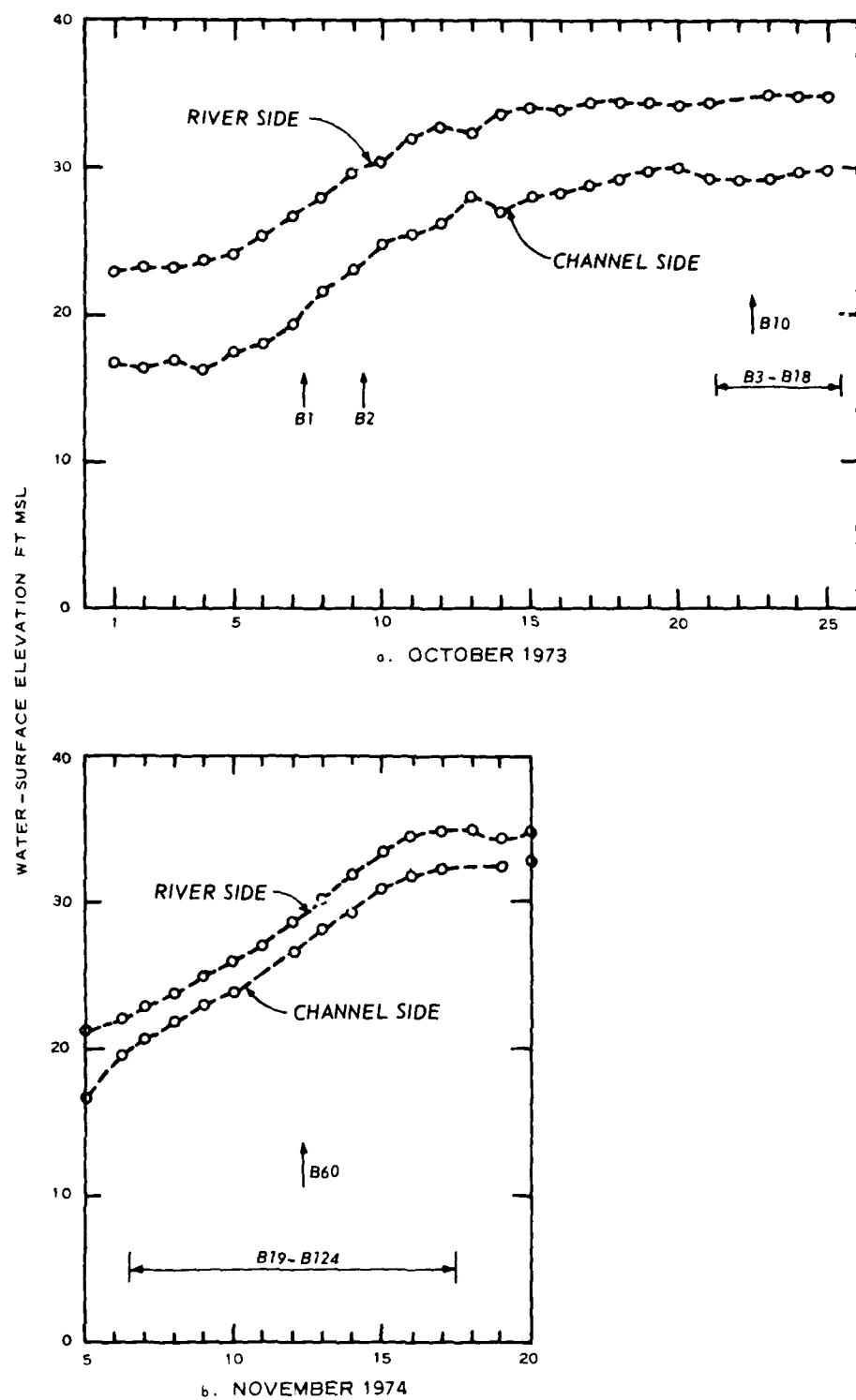
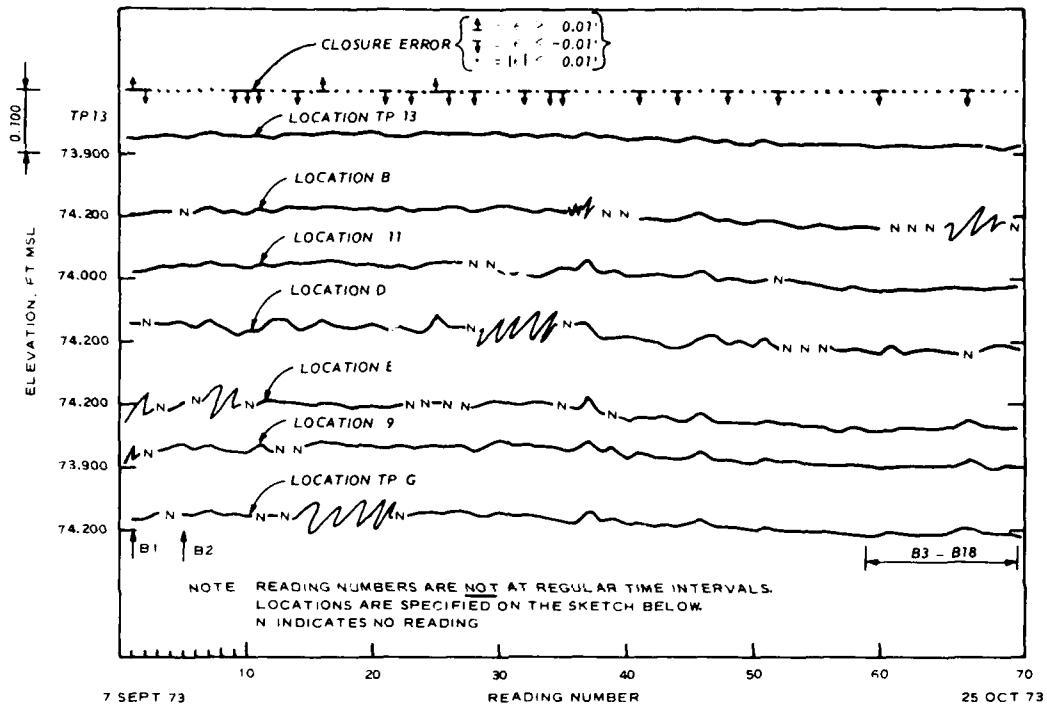
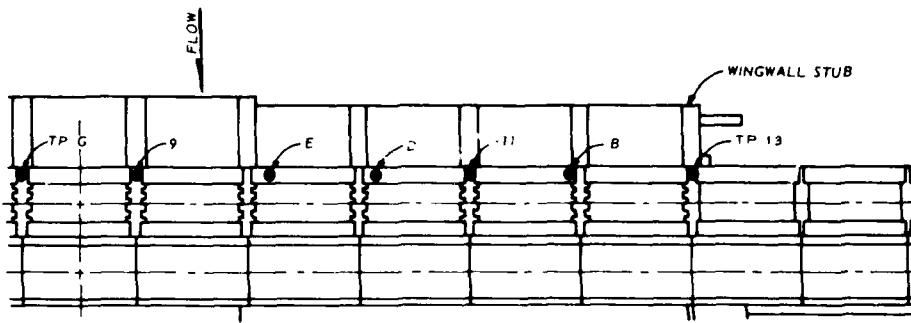


Figure 21. Stages at Old River Low-Sill Structure, October 1973 and November 1974



a. MEASUREMENTS



b. LOCATIONS OF SURVEY MARKERS (PLAN VIEW)

Figure 22. Level data for 7 September-25 October, 1973

were viewed relative to preceding tests inasmuch as the large high-frequency accelerations, which were not expected to be damaging to the structure, masked any low-frequency responses that may have existed in these records. For example, for the first 5-lb charge used in test B10 (Plate 13) the extreme longitudinal acceleration, -0.08 g at location 1112L, is within the acceptable range, -0.1 to $+0.1$ g. On the other hand, the extreme transverse response, -0.13 g at 90T, is outside the acceptable range. The vertical accelerations are, of course, much larger as shown in Plate 14. The signals from transducers 56V and 90V saturate the magnetic tape and are invalid; the large accelerations at 23V, 78V, and 90V are of such a high frequency that they were considered not to represent conditions that would be damaging to the structure. Inspection of the structure following B10 showed no damage and succeeding tests were performed using 5-lb charges.

b. Earth motion. Vertical velocities of the geophones are shown in Plate 14. The initial extreme velocity is about -0.5 in./sec (motion downward) at location 13V1; following this transient motion, a low-frequency oscillation with approximately the following characteristics (13V1) occurs:

Maximum peak-to-peak velocity change, $\Delta V = 0.5$ in./sec
time between positive peaks, $\Delta t = 0.29$ sec

Equivalent frequency, $f = 1/\Delta t = 3.4$ Hz
logarithmic decrement,* $\delta = 2.37$

Equivalent peak-to-peak
displacement, $X = \Delta v/2\pi f = 0.023$ in.

The amplitude of the low-frequency oscillation is reduced at location 13V2 and is not discernible at 13V3. The time of travel between 13V2 and 13V3 of the initial shock is about 0.031 sec (Plate 14); the velocity of the shock wave through the embankment between the transducers is therefore 100 ft/0.031 sec = 3230 fps. As viewed in the field, these data indicated that (a) the low-frequency motion (3.4 Hz) corresponds to a vibration of the structure and foundation, and (b) the damping is high enough so that this motion decays rapidly.

c. Pressures. The records from P1, P2, P3, and P4 during tests B3-B18 contain physically unrealizable negative pressures as shown in Plate 14 and are invalid; this is probably because of dynamic interference between the enclosing 1-ft-diam steel pipes and the transducer

* $\delta = \ln (V_1/V_2)$ where V_1 and V_2 are successive maximum velocities; $\delta = \ln (0.25/0.19)$ for 13V1, test B10 shown in Plate 14.

mounts. As viewed in the field, the data from P5 and P6 for tests B6-B18, and P1 and P7 for tests B19-B124, indicated that the magnitude of the water shock is highly dependent not only on size of charge but on numerous additional conditions. Some of these additional factors are (a) the point of attachment of the charge to the wing wall, (b) the manner in which the charge is attached to the wing wall, and (c) the number of and the time between successive explosions.* These conditions, other than item c, were not well documented in the field and are not presented herein. Only the maximum pressures, P5 in Plate 14, were of value in the field (although limited due to items (a), (b), and (c), above) for assessing the consequence of varying the charge size.

Data Reduction and Analysis

41. Two sets of 1-sec digital playbacks are shown in Plates 15 and 16 for the first explosion in test B10, and the single explosion in test B60, respectively. The data are digitized at 1000 samples per second and folded at 500 Hz. The corresponding FFT's, evaluated as noted in Part II, are presented in Plates 17 and 18 (longitudinal data and transverse and vertical data, respectively, for test B10) and in Plates 19 and 20 (longitudinal and transverse data and vertical data, respectively, for test B60). The FFT's are evaluated in steps of 1 Hz and extend to 500 Hz; the logarithmic frequency scale, rather than the linear scale used previously, is required so that frequencies corresponding to the initial transient movement (high frequency) as well as the subsequent rocking motion of the structure (low frequency) are shown. A summary of the significant frequency components from Plates 17-20 and the corresponding extreme accelerations from Plates 15 and 16 are listed below. Also listed are the peak-to-peak displacements of a comparable simple sinusoidal motion (Equation 1).

* For example, during test B10 (Plate 14), the first 5-lb charge results in a shock pressure of about 900 psi at location P5; the second 5-lb charge results in a pressure of only about 300 psi.

Test	Trans- ducer	Acceleration, g's				Corresponding Frequencies,* Hz			Equivalent Displacements, in		
		Initial Transient		Rocking Motion		Initial	Rocking	Initial	Rocking	Initial	Rocking
		Maximum	Minimum	Maximum	Minimum	Transient	Motion	Transient	Motion	Transient	Motion
B10**	23L	0.0058	-0.0050	0.0038	-0.0021	10,D	4	0.0011	0.0036		
	56L	0.045	-0.031	0.013	-0.010	30,100D	4	0.0008	0.014		
	78L	0.028	-0.025	0.020	-0.008	12,D	4	0.0036	0.017		
	† 90L	0.065	-0.054	0.017	-0.019	10,D	4.5	0.0116	0.017		
	† 1112L	0.075	-0.055	0.022	-0.020	D	4	--	0.0257		
	† 1112L2	0.022	-0.036	0.008	-0.008	D	3.5	--	0.0174		
	56T	0.043	-0.047	0.010	-0.008	42,98D	4	0.0005	0.0110		
	90T	0.131	-0.088	0.014	-0.010	44,D	4	0.0011	0.0147		
	23V	0.18	-0.17	--	--	334N	NCP	--	--		
	78V	0.65	-0.59	--	--	53,N	NCP	0.0043	--		
B60	† 1112V	0.65	-0.55	--	--	76,N	4	0.0020	--		
	56L	0.013	-0.014	0.012	-0.011	D	5,8	--	0.0090		
	78L	0.024	-0.024	0.023	-0.023	D	6	--	0.0125		
	90L	--	--	0.100	-0.065	D	6,8	--	0.0449		
	1011L	--	--	0.096	-0.078	D	6,8	--	0.0473		
	56T	0.028	-0.027	0.008	-0.008	44,D	5	0.0003	0.0063		
	90T	0.083	-0.127	0.012	-0.008	63,D	5	0.0005	0.0078		
	56V	1.040	-0.975	--	--	233N	NCP	0.0004	--		
	78V	1.272	-0.968	--	--	252N	NCP	0.0003	--		
	89V	1.488	-1.557	--	--	218N	NCP	0.0006	--		
** Transducers 56V and 90V saturate the magnetic tape and are invalid; these data are not digitized.	90V	1.914	-2.178	--	--	163N	NCP	0.0015	--		
	1011V	3.587	-3.839	--	--	162N	NCP	0.0028	--		

* N and D apply to high frequencies (over 10 Hz); they indicate "noise" (many peaks) and signal "damped" to low values, respectively. NCP indicates "no clear peak" for low frequencies (less than 10 Hz). Where two frequencies are listed, the underlined one is used in the displacement calculation.

** Transducers 56V and 90V saturate the magnetic tape and are invalid; these data are not digitized.

† The initial transient peak is outside the digitized time period for these four signals; the extreme transient accelerations are from the analog playbacks shown in Plates 13 and 14.

Discussion of Results and Comparison with Previous Values

42. The low-frequency displacements during the demolition activities are substantially larger than any of the preceding tests; for example, the maximum displacement at 1011L in test B60 is nearly fifty times as large as any previously observed movement (0.05 in. compared with 0.001 in.). However, the short duration of the response to the explosions probably permits a substantially larger movement to be tolerable than the movement during continuous vibration as in the earlier tests. The following changes occur between the two demolition activities.

- a. The dominant low frequency is higher during the later series (6 Hz as compared with 4 Hz); one factor that tends to make the structure more rigid in the later series is that the cavity beneath the structure has been filled with grout during the interim between the two series of tests.
- b. The low-frequency displacements are largest during the later series; the major factor is the shorter distance (see Figure 16) between the structure and segments D and E (about 50 ft) and between the structure and segments B and C (about 120 ft).

PART V: LOW-SILL GATE LOADS (MAY-JUNE 1975)

43. Dogging devices (Figure 23) were installed in the gate slots of the Old River Low-Sill Control Structure to support the gates; however, the devices were not designed to operate the gates in a partially submerged position.¹⁸ Instead, the original plan was for the individual gate bays to have either all the gate leaves removed or completely closed.

44. During 1973-1975, the need for additional control became evident; hydraulic model tests¹⁹ were performed to help determine if overloading would occur during partial gate closure using the existing dogging devices. Field data were required to complete the study. Consequently, the prototype tests were conducted to obtain information regarding the loadings that would occur on the dogging devices during partial gate closure.

Gate Hoist System

45. A gantry crane (Figure 24) is used to manipulate the gates. The crane is equipped with two lifting blocks (Figure 25) which attach to the top of the gate leaves. To keep the two lifting blocks at the same elevation, the cable arrangements controlling each block are connected by an equalizer bar (Figures 26 and 27) located near the crane bridge elevation.

46. Each end of a leaf is supported in the gate slot by a dogging device for a partial gate closure (Figures 23 and 25); the devices are all at el 69.0 ft msl. Various combinations of gate leaves can be supported by one set of dogs making use of the 2.5-ft-diam wheels on the ends of the leaves. The complete gate in each of the eight shallow bays is made of three leaves; in the three deep bays, four leaves are required. Leaves not in the gate slot are stored in special racks alongside the gate slot (Figure 25).

47. The test program required determination of:

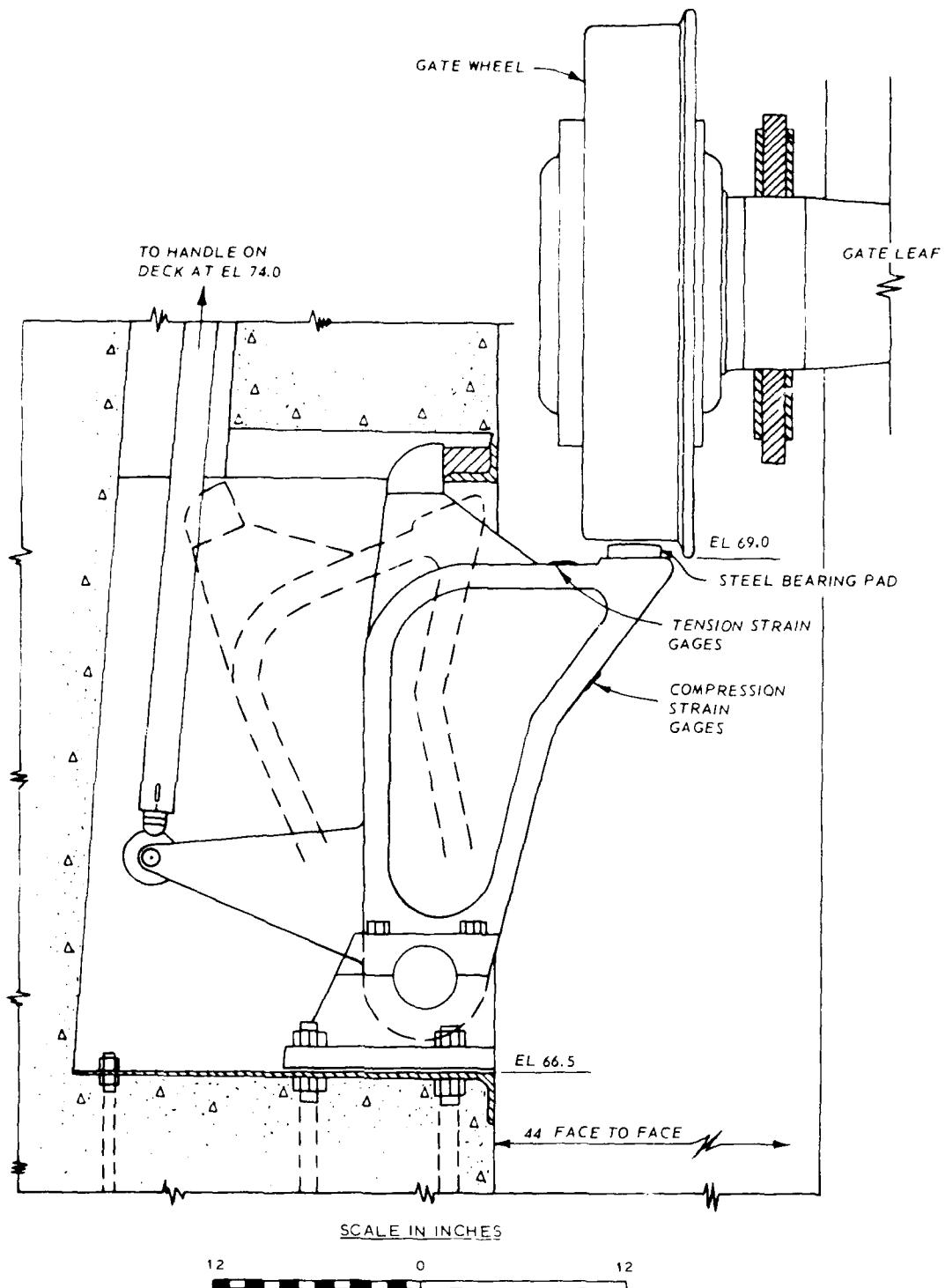


Figure 23. Gate dropping device



Figure 2a. Gantry crane used to manipulate gate leaves

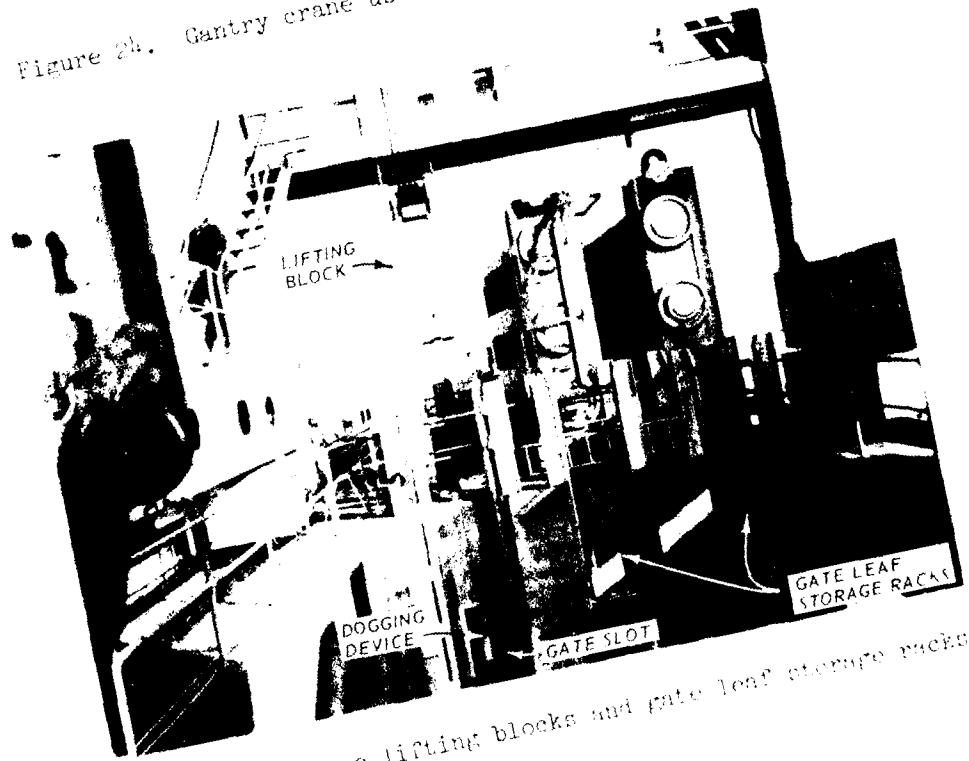


Figure 2b. Crane lifting blocks and gate leaf storage racks

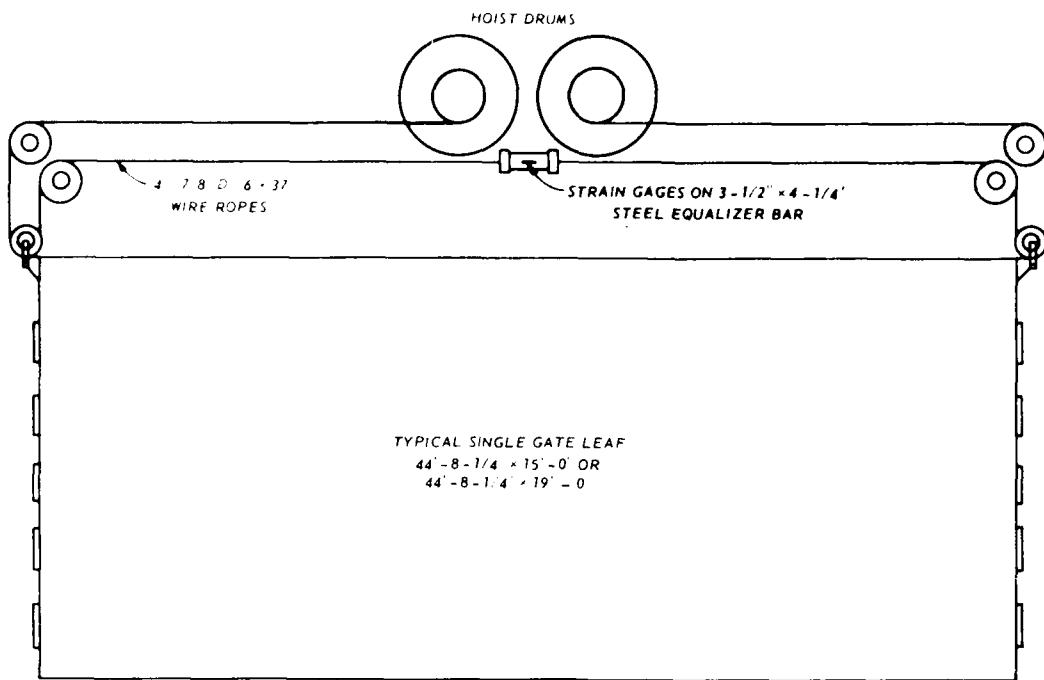


Figure 26. Gate hoist schematic

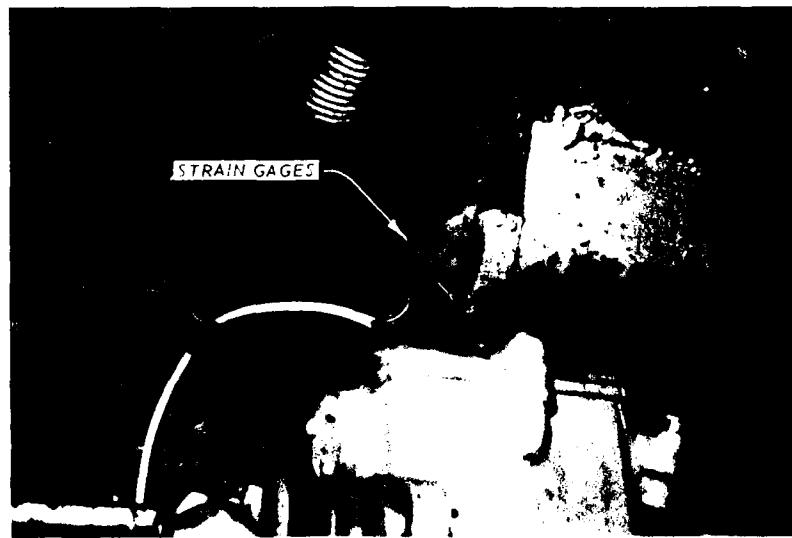


Figure 27. Crane equalizer bar (3-1/2 in. by 4-1/4 in.) with strain gages installed

- a. The load experienced by the dogging devices during periods of operation.
- b. The temperature of the gate during operation.
- c. Vibration of the installed gate leaf and the piers of the structure.

Instrumentation

48. Load-supporting members at three locations were fitted with foil strain gages. One set of four gages was installed on each of the two dogging devices in bay 6 as shown in Figure 28. Convex steel pads were tack-welded to the dogging devices (Figures 23 and 28) to assure the gate weight was applied as a point load. Another set of four gages was installed on the equalizer bar in the crane as shown in Figure 26.

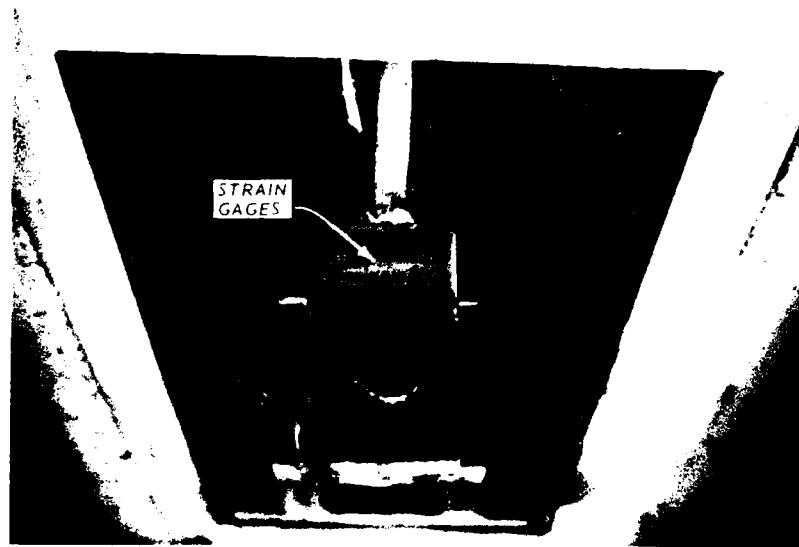
49. Two load cells (one at each side, Figure 29) were used to weigh the gates while the gates were suspended in air by the crane. The load cells had a range of 50,000 lb each.

50. A portable accelerometer arrangement (Figure 30) was used to measure gate and structural vibrations. The accelerometers were mounted such that vertical accelerations were measured using an accelerometer having a range of ± 2.5 g's peak-to-peak and accelerations in the horizontal plane were measured using accelerometers having a range of ± 0.25 g.

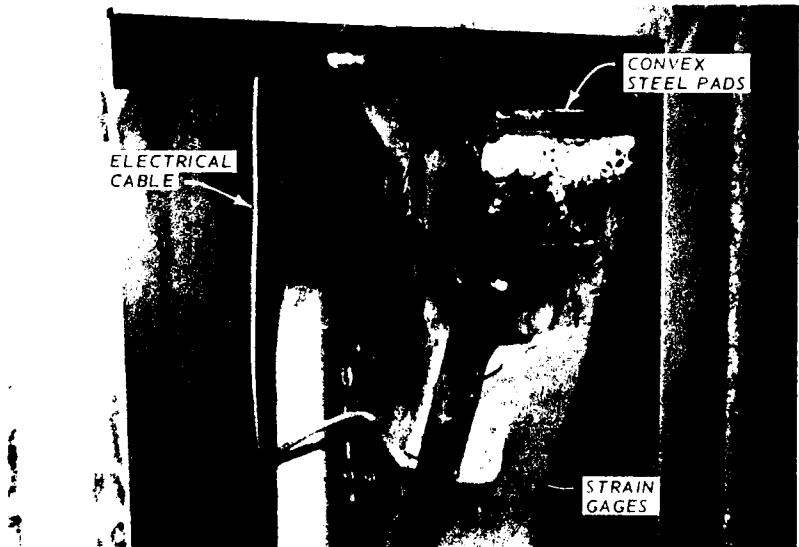
51. A remote temperature sensing device was used on the top gate leaf in bay 6 to record gate temperature (Figure 31) during a portion of the tests.

52. The strain gages and temperature sensor were connected to strip chart recorders (Figure 32) which were capable of recording two channels simultaneously. The accelerometer group was connected to a direct print strip chart recorder (Figure 32).

53. The following is a summary of the instruments used during the test program:



a. Top view



b. Frontal view

Figure 28. Deforming device with strain gages installed



Figure 29. Load cell used to weigh gates

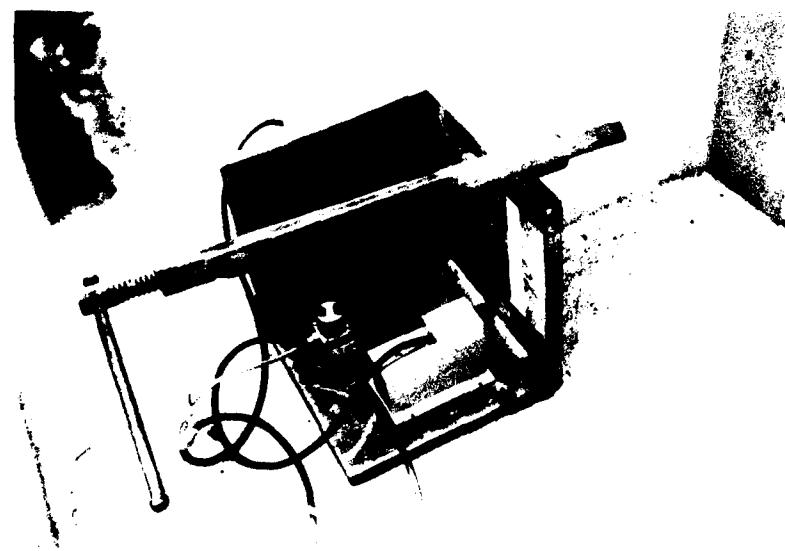


Figure 30. Portable accelerometer group

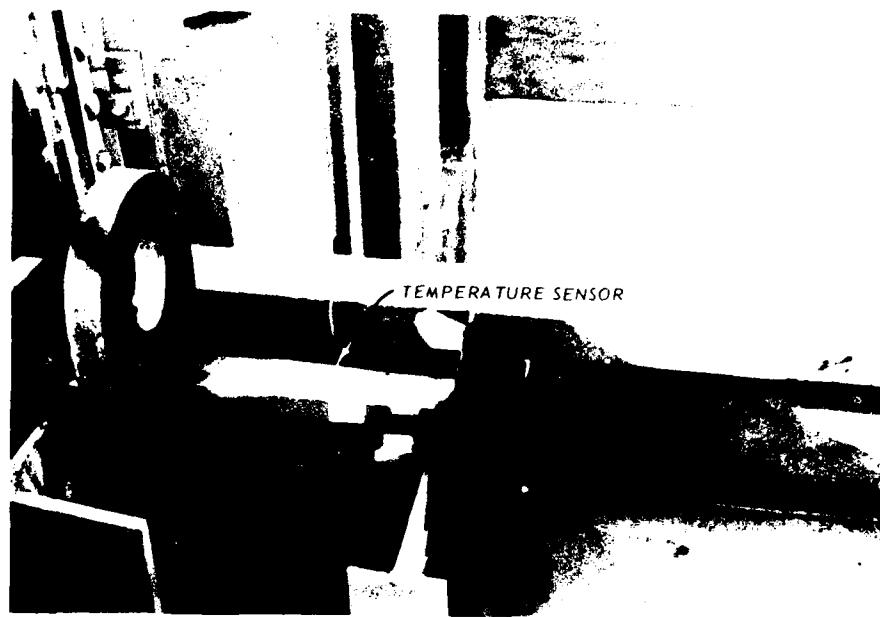


Figure 31. Temperature sensor located on skin plate of μ te

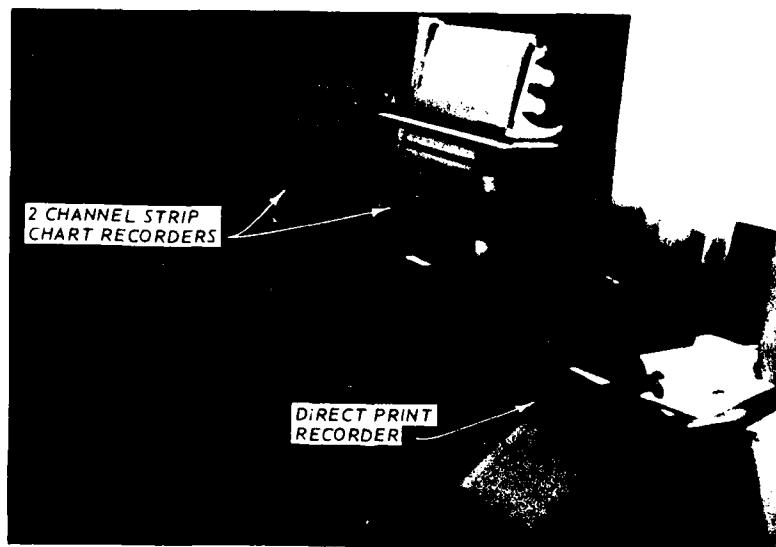


Figure 32. Electronic recording equipment

Instrument			
Name	Quantity	Location	Purpose
Foil strain gage	8	(a) Crane equalizer bar (Figures 26 and 27) and (b) gate dogging device (Figure 23)	(a) Evaluate total gate loads and (b) measure strain at key locations in the gate support mechanism
Load cell	2	Crane lifting blocks (Figure 29)	Measure weight of individual gate leaves
Accelerometer arrangement (3 strain gage accelerometers)	1	Piers on north sides of bays 3, 5, 6, 7, and 8 and on Gates 5, 6, and 7 (Figure 30)	Measure vibration of gates and piers
Thermistor	2	Top gate leaf in bay 6 (Figure 31)	Measure gate leaf temperature
Direct-print oscillosograph recorder	1	Instrument trailer (north end of structure)	Record acceleration data
2-Channel, electric stylus, strip chart recorder	2	Instrument trailer (north end of structure)	Record strain gage and temperature data

Gate Weights

54. To obtain accurate gate weights, each crane lifting block was fitted with a 50,000-lb load cell and a cable sling. The sling was attached to the gate and the gate was suspended in air and weighed by the load cells (Figure 29). The resulting weights are:

Gate Number	Gate Weight, kips		
	Left Side South	Right Side North	Total
1-1W	26.2	26.1	52.3
1-2W	42.1	42.0	84.1
1-3W	44.2	44.7	88.9
7-1L	26.6	26.3	52.9
6-2L	36.4	38.4	74.8
6-3L	44.9	48.9	93.8
7-4L	35.4	37.1	72.5

Strain Gage Calibration

55. To obtain meaningful data from the strain gages installed in the crane and to confirm the linearity of the strain-load relationship in the crane equalizer bar, the system was calibrated as follows. Gates of known weight were lifted with the crane, and indicated strains were recorded with a strip chart recorder. The pen deflection on the strip chart varied with the total strain in the gages. The calibration was plotted as a load (gate weight) versus recorder pen deflection curve as shown in Figure 33.

56. The calibration procedure for the strain-gaged dogging devices was similar to the procedure used to calibrate the equalizer. Calibration data were obtained by placing gates of known weight and combinations of those gates on the dogging devices and recording the strain gage output. The calibration curves obtained from these data are presented in Figure 34. Each load was assumed to be carried equally by each dogging device.

Crane Equalizer-Bar Tests

57. Gate loads for control structure operation were determined by means of the equalizer-bar strain gages for varying operating conditions in each of the 11 bays of the structure. The various conditions under which these tests were conducted are shown below.

<u>Bay No.</u>	<u>Gate Opening ft</u>	<u>Head Range ft</u>		<u>Range of Upstream Stage, ft</u>		<u>Number of Tests</u>
		<u>High</u>	<u>Low</u>	<u>High</u>	<u>Low</u>	
1	28.9	7.8	4.6	49.4	43.2	10
2	28.9	7.8	4.6	49.4	43.2	10
3	28.9	7.8	4.6	49.4	43.2	10
4	28.9	7.8	4.6	49.4	43.2	10
5	43.4	6.7	6.4	53.2	52.6	3
5	28.8	6.2	5.9	50.5	49.9	10
5	24.8	7.7	4.4	49.9	43.1	46

(Continued)

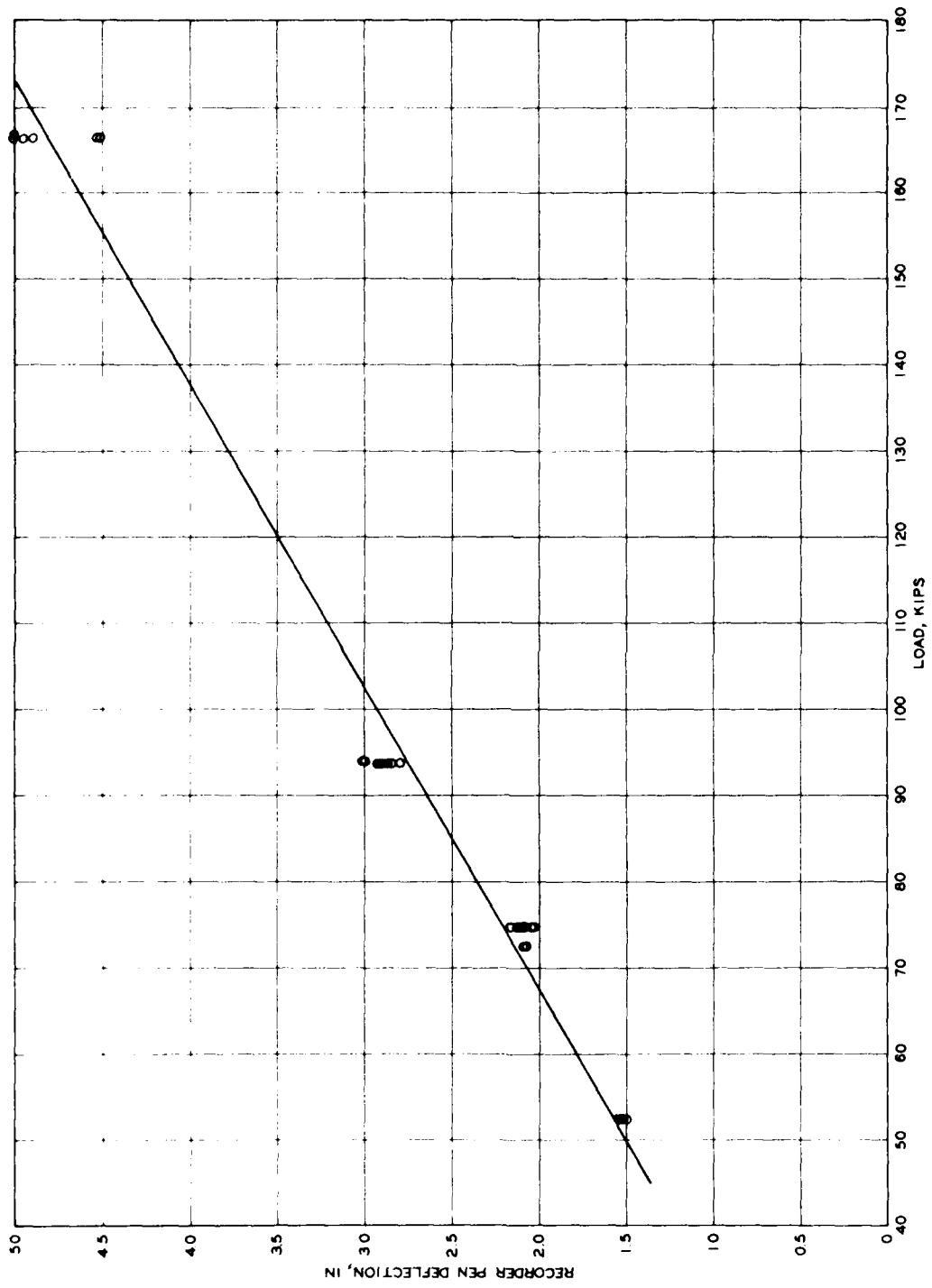


Figure 33. Crane equalizer bar calibration curve

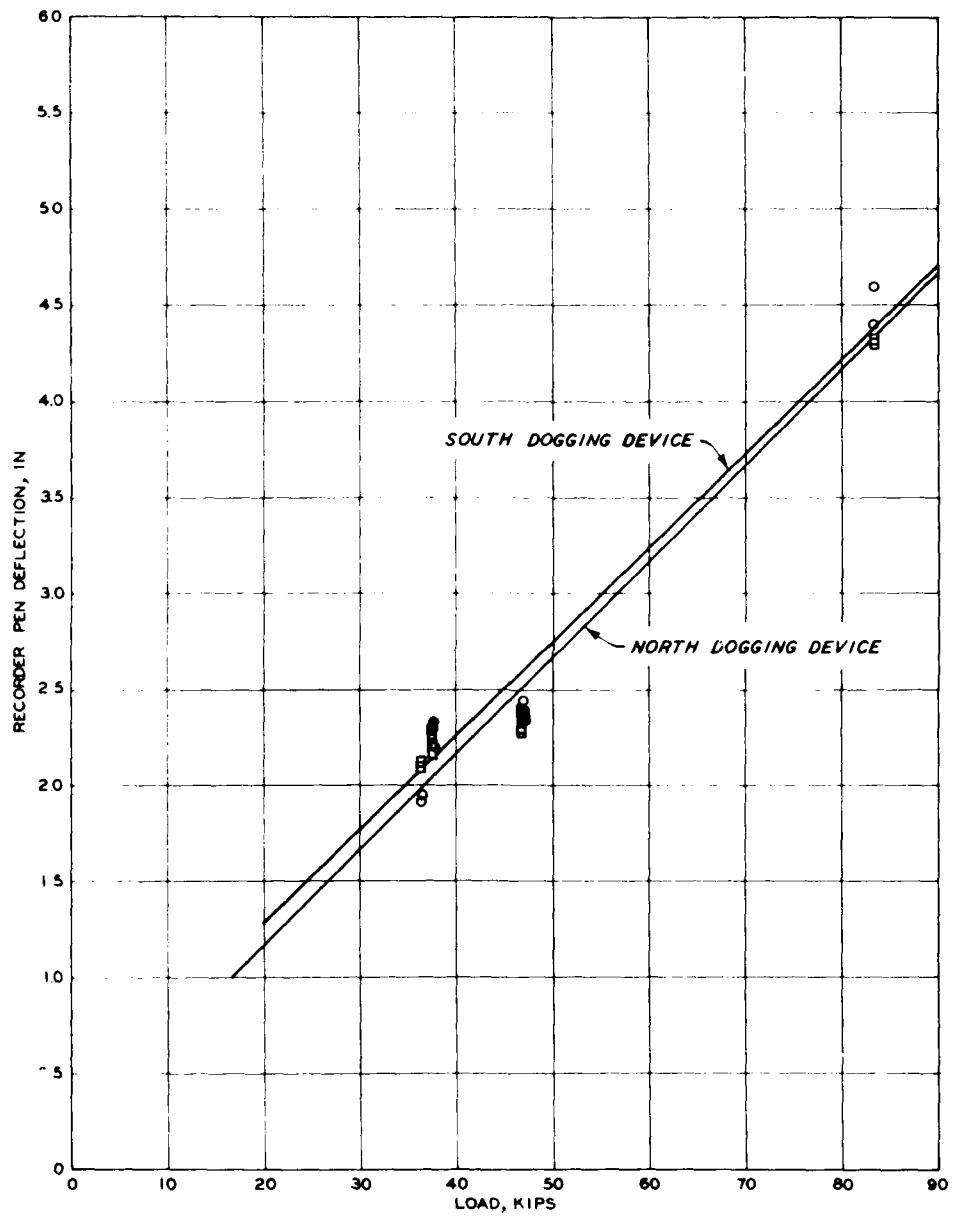


Figure 34. Dogging device calibration curves

Bay No.	Gate Opening ft	Head Range ft		Range of Upstream Stage, ft		Number of Tests
		High	Low	High	Low	
6	43.4	7.2	6.1	54.1	52.0	7
7	43.4	7.1	7.1	54.0	54.0	1
7	28.8	6.2	5.9	50.7	49.9	8
7	24.8	7.8	4.5	50.0	43.0	46
8	28.9	7.8	4.6	49.4	43.2	10
9	28.9	7.8	4.6	49.4	43.2	10
10	28.9	7.8	4.6	49.4	43.2	10
11	28.9	7.8	4.6	49.4	43.2	10

58. Prior to each test, the crane was positioned so that the lifting blocks and cable were suspended in air with no load attached. This condition established the "crane zero" record on the strip chart as shown in Figure 35. When the test was completed, the apparatus was returned to the crane zero position to determine if a "zero drift" had occurred. Such drifts were small, as shown in Figure 35, and were taken into consideration during data reduction.

Dogging Device Tests

59. A zero loading was established prior to each test and following each test by removing all external loads. Following the zeroing procedure, the gate configuration being tested was placed on the dogging devices. The indicated strains were recorded on a strip chart (Figure 36).

60. Gate loads being transmitted to the dogging devices in bay 6 were recorded for various operating conditions and for varying time intervals. The following is a list of the conditions under which these tests were conducted.

Gate Opening, ft	Head Range, ft		Upstream Stage, ft		Number of Tests
	High	Low	High	Low	
43.4	7.1	6.2	54.0	51.6	10
28.8	6.2	6.1	51.5	50.6	3
24.8	7.8	4.7	50.0	43.3	17

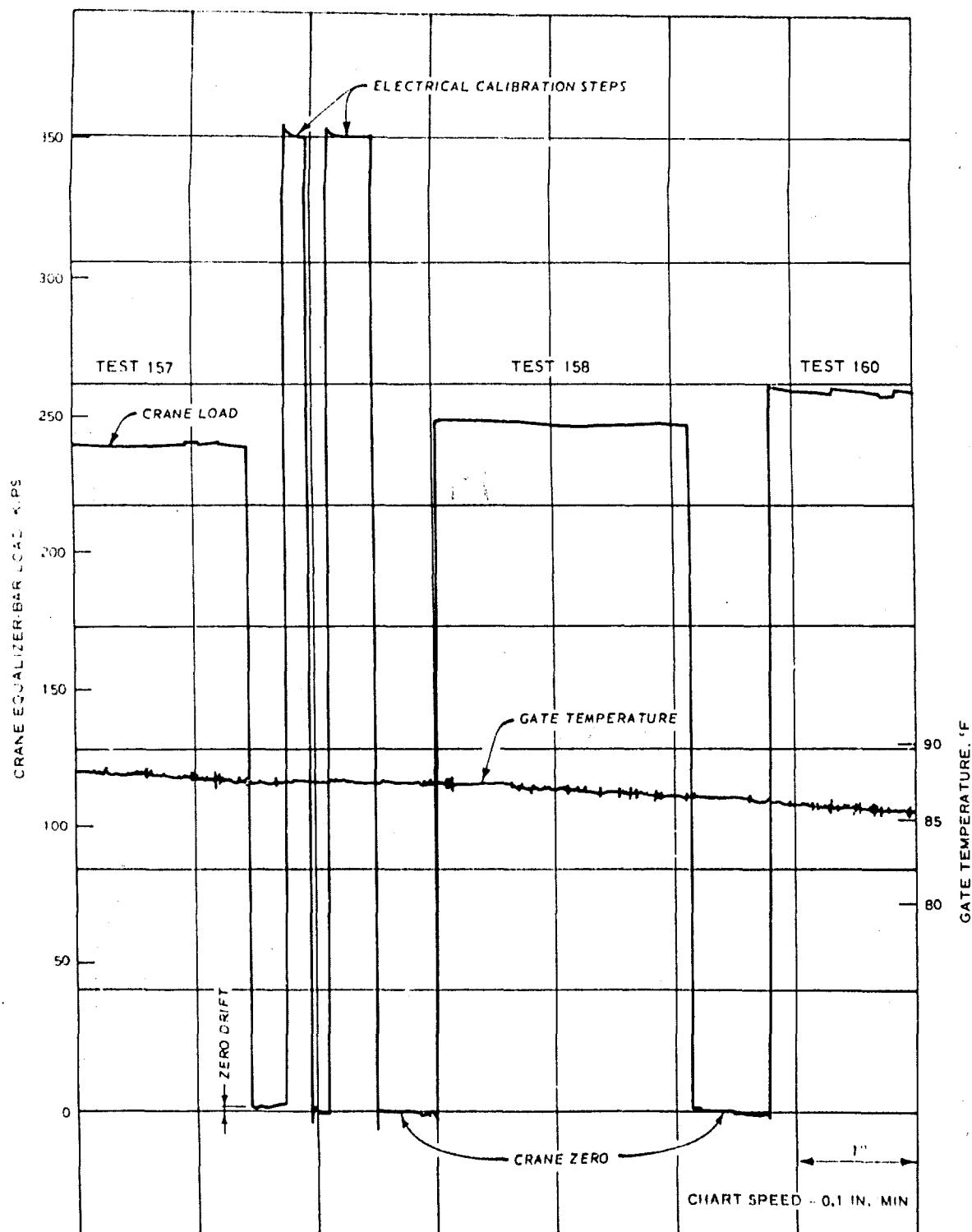


Figure 35. Typical crane equalizer-bar load and gate temperature record

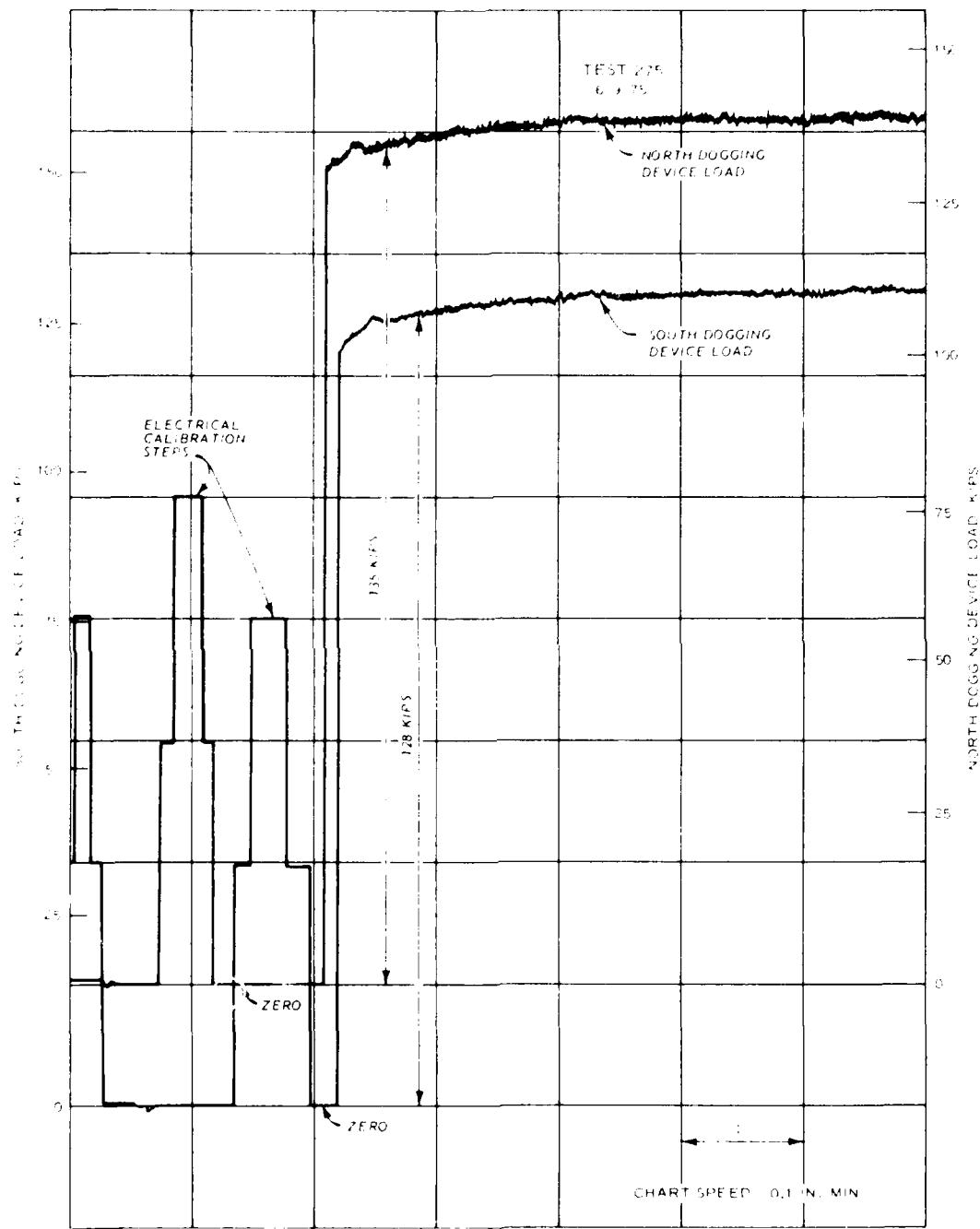


Figure 36. Typical dogging device test record

Acceleration Measurements

61. Gate and structural accelerations were monitored during differing test conditions and at various positions on the structure and the gates. Accelerations of the structure were measured on the piers at the north side of bays 3, 5, 6, and 8 and on the south side of bay 6 (Figure 37). Gate accelerations were measured near the center of the top of the uppermost gate leaf in bays 5, 6, and 7 (Figure 38).

Summary of Measurements

62. The following tabulation summarizes the types of data obtained.

<u>Test to Determine</u>	<u>Test Equipment</u>	<u>Data Obtained</u>	<u>Test Numbers</u>
Gate weights	Load cells	Gate weights	--
Crane calibration	Crane	Strain	--
Dogging device calibration	Dogging device	Strain	--
Total gate loads, bays 1-11	Crane	Total gate loads	6,9,14,16,18,20,21, 23,24,26,28,42-53, 55,57,58,60-62,68, 71-78,87,88,93,96- 104,114,117-119,122- 126,135,138-140,143- 148,157,160-163, 166-170,179,180, 183-189,202-210, 219,222-228,240- 247,257,258,261- 264,273,276
Total gate loads, bay 6	Dogging devices	Total gate loads	7,8,13,13A;15,17,19, 22,25,29,32,40,40A, 67,89
Total gate loads and temperature effects	Dogging devices	Total gate loads and gate temperature	90-92,95,116,121,137, 142,159,164,182, 221,239,260,275

(Continued)

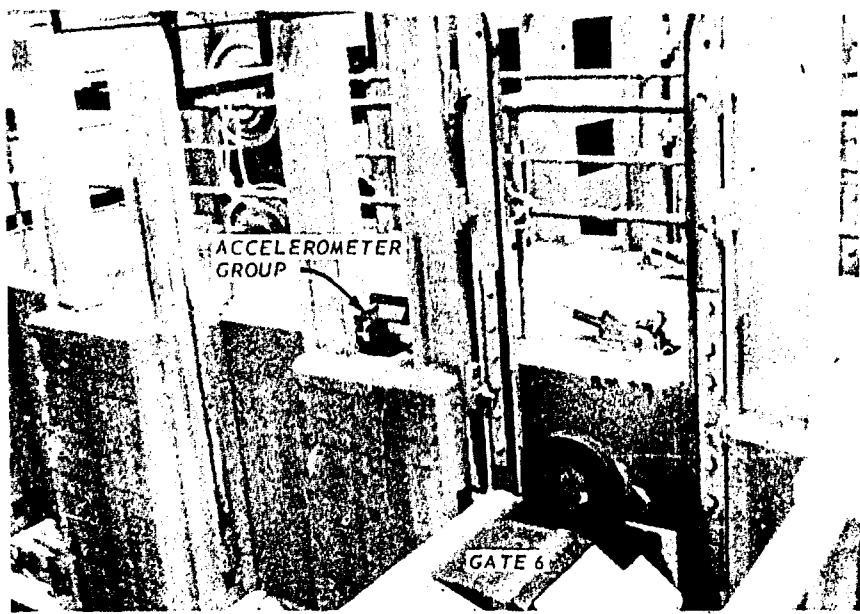


Figure 37. Accelerometer group located on pier

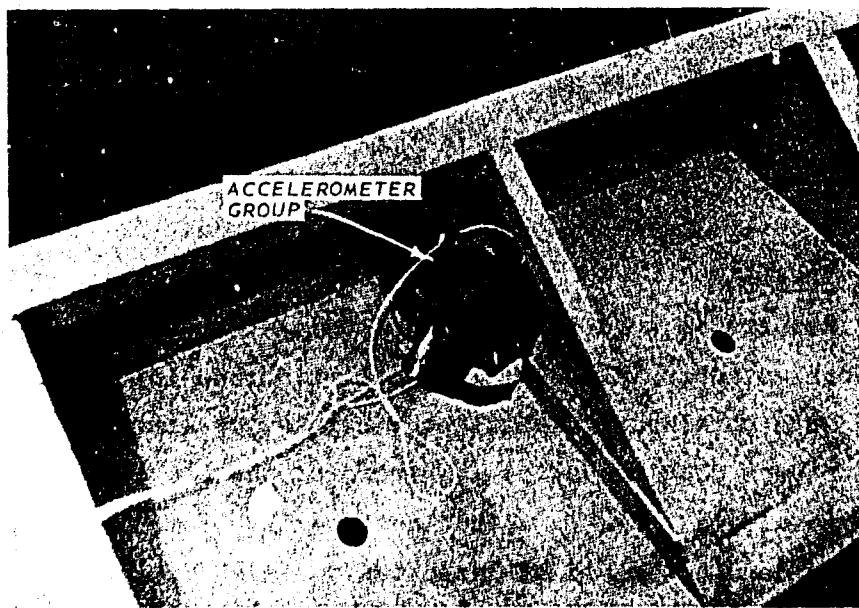


Figure 38. Accelerometer group located on top of top leaf of gate

<u>Test to Determine</u>	<u>Test Equipment</u>	<u>Data Obtained</u>	<u>Test Numbers</u>
Vibration of piers	Accelerometers	Accelerations	N62, S65, N58, N69, N816, N617, N623, N631, N639, N643, N344, N649, N350, N654, N355, N660, N361, N363, N664, N668, N371, N373, N676, N381, N384, N685, N687
Vibration of gates	Accelerometers	Accelerations	C670, C778, C679, C580, C674, C575

Results

Crane data

63. Data acquired during the test period using the crane are presented in Tables 3-6. These data include head on the gate and upstream river stage. The upstream river stage was measured at a staff gage about 100 yd north of the structure. Using the total load experienced by the crane and the known weight of the gate configuration being tested, the resulting net load, attributed to hydraulic effects, was determined and is also presented in these tables. The net loads in bays 5, 6, and 7 ranged from 17.9 kips of downpull in bay 5 to 15.4 kips of uplift in bay 7. The net loads in bays 1-4 and 8-11 ranged from 11.2 kips of downpull in bay 8 to 13.0 kips of uplift in bay 3.

Dogging device data

64. Results of load data obtained using the dogging devices in bay 6 are presented in Table 7. The total high and low gate loads shown in Table 7 were obtained by summing the highest or lowest average readings for each dogging device. Therefore, the two loads summed to obtain the totals in Table 7 did not necessarily occur at the same time. Although this does not yield an instantaneous high or low, in the case of the high loads it is a conservative number. This table also contains head and upstream stage information recorded during the tests. The total apparent dogging device loads varied during the tests from 233.3 to 162.6 kips for a gate opening of 43.4 ft and using a gate made up

of gate leaves 6-3L and 7-4L, which have a total dry weight of 166.3 kips. These apparent loads varied from 298.9 to 265.0 kips for a gate opening of 28.8 ft using gate leaves 6-2L, 6-3L, and 7-4L, which have a total dry weight of 241.1 kips. The loads varied from 300.1 kips to 240.4 kips at a gate opening of 24.8 ft using the 241.1-kip gate leaf combination. The net or difference in the dry gate weight and apparent dogging device loads is not assumed to be a result of hydraulic forces alone as will be discussed in subsequent paragraphs.

Acceleration data

65. A representative sample of the acceleration data recorded during the test was reduced and the results are presented in Table 8. These data show structural accelerations in the vertical direction varied in frequency from 392 to 236 Hz. Maximum peak-to-peak accelerations varied from 0.121 to 0.007 g and displacements, calculated using Equation 1, varied from 8.48×10^{-7} to 6.34×10^{-8} ft. In the upstream-downstream direction, the frequency varied from 23 to 9 Hz, the maximum peak-to-peak accelerations from 0.0074 to 0.0011 g, and the displacement from 4.40×10^{-5} to 3.67×10^{-6} ft. In the directions transverse to the flow, frequencies varied from 21 to 14 Hz, maximum peak-to-peak accelerations from 0.0123 to 0.0032 g, and displacements from 4.87×10^{-5} to 1.02×10^{-5} ft. These data also show gate accelerations varied in frequency from 261 to 115 Hz, from 22 to 14 Hz, and from 20 to 16 Hz for the vertical, upstream-downstream, and transverse directions, respectively. The displacements varied from 2.19×10^{-5} to 6.57×10^{-7} ft, from 5.83×10^{-5} to 1.50×10^{-5} ft, and from 2.50×10^{-4} to 6.46×10^{-5} ft in the vertical, upstream-downstream, and transverse directions, respectively.

Temperature and Duration Effects

66. Strain gage data obtained from the dogging devices indicated that the recorded loads were influenced by the length of the test and temperature changes during the tests. Figure 36 indicates an increase in load in the first few minutes of the test. The load experienced by

the crane showed no change due to temperature changes (both air temperature and gate temperature) & i no change in load due to the length of the test period if hydraulic conditions did not vary. Plates 21 and 22 show that the changes in apparent dogging device load occurring beyond the initial increase are related to the gate temperature. The effects of temperature change are much greater than those resulting from head and upstream stage changes. A method of calculating temperature and stress distributions in bridges is presented by Hunt and Cooke.²⁰

67. Test 282 (Plate 23) was conducted in order to verify the effect of temperature on apparent dogging device loads. This test was conducted during a wide range of temperatures while a 74.8-kip gate was suspended by the logging devices in the gate slot so that no portion of the gate was in the water. It was determined that the apparent load ranged from 79.1 to 109.8 kips. This 30.7-kip change in apparent load represents 41 percent of the actual gate weight. Plate 21 also shows that the apparent gate load is inversely related to the temperature of the gate.

68. The relationship of apparent gate load to temperature may be due to expansion and contraction of the gate leaf positioned on the dogging devices. Frictional forces at the point of contact between the gate and the dogging devices would allow forces caused by expansion and contraction of the gate to be transmitted to the dogging devices; forces so transmitted would yield false load information. These forces would not be vertical and would cause deformations of the dogging devices differing from forces of the same magnitude acting vertically, but could appear to the strain gages as possible changes in the vertical load.

Conclusions

69. Forces acting on the dogging devices during the tests were affected primarily by gate weight, by hydraulic forces, and temperature changes. Gate weight had the greatest effect on the forces; next greatest were temperature changes and then hydraulic forces. Gate weight, hydraulic forces, and temperature changes should be considered in future operation of the structure.

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Table 1
Demolition Sequence, Wing-Wall Segments B and C

Test	Date	Time	Charges* Placed		Audible** Explosions	Wall Segment	Additional Notes
			No.	Weight, lb			
B1	Oct 1973	1500	2	10	1	C	No water-shock pressures or ground-velocity measurements; magnetic tape saturates on accelerometer (vertical) channels. The charges are placed in a line along the base of segment C
B2	7	1455	2	5	1	C	No water-shock pressures or ground-velocity measurements; duplicate vertical acceleration measurements at two locations; no measurements on Gate 7
B3	21	1125	3	5	3	C	No ground-velocity measurements; major change in accelerometer locations
B4	21	1246	3	5	3	C	No ground-velocity measurements
B5	21	1409	4	5	3	C	First complete data set
B6	21	1610	3	5	3	C	No unusual occurrences
B7	22	1003	3	5	3	C	No unusual occurrences
B8	22	1439	6	5	6	C	Diver cuts exposed reinforcement bars prior to placing these charges
B9	22	1650	8	5	6	C	Crane overturns on bridge deck immediately prior to detonation
B10	22	1739	2	5	2	C	No unusual occurrences
B11	23	1158	7	5	8 or 9	C	Segment C of wall collapses during this series of explosions
B12	23	1456	6	5	6	B	These charges were placed along a crack located well above the rock line, the lower section of segment B is removed later
B13	23	1615	6	5	3	B	No unusual occurrences
B14	24	0957	6	5	5	B	No unusual occurrences
B15	24	1210	6	5	5	B	The exposed section of segment B collapses. Diver damage-survey follows this blast
B16	25	1000	3	5	1	B	These charges initiate demolition work on the base of segment B; i.e., the upper portion has been removed
B17	25	1236	5	5	5	B	These explosions finish the demolition work on the base of segment B
B18	25	1616	2	5+	1	C	This explosion removes vertical splinter left previously at the east end of segment C. Hydrographic survey (NOD: launch-mounted fathometer for depth measurement and ranges for positioning) follow this explosion; preliminary measurements indicate that portions of the concrete are still above grade

* These values were not confirmed by the writers; they were provided by the demolition contractor, Mr. Patrick Kenny, Technical Explosives. The detonation sequence is from west to east, i.e., the charges nearest the structure are detonated first. Line charges were used during B1 and B2; point charges were used for the balance of the demolition program. The explosive is C-4.

** Delay times are evident in the oscillograph playbacks; fewer explosions than charges placed indicate either simultaneous detonations, or no detonation, of some of the charges. Accidental simultaneous detonations did occur, apparently because currents occasionally caused the primacords to become entangled.

† These charges are placed on the south face of the pertinent segment well above the rock line; the purpose is to cause the segment to overturn in a northerly direction.

Table 2
Demolition Sequence, Wing-Wall Segments D and E

Test	Date Nov 1974	Time, hr	Charge Used lb	Additional Notes
B19	7	1600	2.5	Gates 7-11 closed
B20	7	1651	2.5	All gates open
B21	8	1313	2.0	Gates 7-11 closed
B22	9	0957	2.0	Gates 7-11 closed
B23	9	1351	2.0	Gates 7-11 closed
B24	9	1403	2.0	Gates 7-11 closed
B25	9	1415	2.0	Gates 7-11 closed
B26	9	1441	2.0	Gates 7-11 closed
B27	10	1046	2.0	Gates 6-11 closed*
B28	10	1110	2.0	
B29	10	1128	2.0	
B30	10	1130 (est)	2.5	
B31	10	1249	2.0	
B32	10	1354	2.0	
B33	10	1427	2.5	
B34	10	1451	2.0	
B35	10	1522	2.0	
B36	10	1540	2.0	
B37	10	1650	2.0	
B38	11	1013	2.0	
B39	11	1043	2.0	
B40	11	1110	2.5	
B41	11	1129	2.5	
B42	11	1231	2.0	
B43	11	1250	2.0	
B44	11	1313	2.0	
B45	11	1343	2.0	Groove 3-5 in. deep complete along base of wall
B46	11	1418	2.0	
B47	11	1534	2.0	B46 and B47 are tube and split charges, respectively.
B48	11	1552	2.0	Decision is made to use tube charges
B49	12	1022	2.0	Reinforcing is exposed
B50	12	1041	2.0	
B51	12	1059	2.0	
B52	12	1116	2.0	
B53	12	1219	2.0	
B54	12	1234	2.0	
B55	12	1254	2.0	
B56	12	1331	3.0	
B57	12	1400 (est)	2.0	
B58	12	1425	2.0	
B59	12	1456	4.0	
B60	12	1530	5.0	
B61	13	0946	4.0	Tape recorder off-line
B62	13	1020	4.0	Tape recorder off-line
B63	13	1052	4.0	Tape recorder off-line
B64	13	1116	4.0	Tape recorder off-line
B65	13	1139	4.0	Tape recorder off-line
B66	13	1234	4.0	No data
B67	13	1255	4.0	Recorder back on-line
B68	13	1316	5.0	

(Continued)

* Gates 6-11 remain closed to end of Test B91; Gates 5-11 closed thereafter.

Table 2 (Concluded)

Test	Date Nov 1974	Time, hr	Charge		Additional Notes
			Used	lb	
B69	13	1338		4.0	
B70	13	1408		4.0	
B71	13	1425		4.0	
B72	13	1440		4.0	
B73	13	1456		4.0	
B74	13	1516		5.0	
B75	14	0848		4.0	
B76	14	0908		4.0	
B77	14	0939		4.0	
B78	14	1000		4.0	
B79	14	1023		4.0	
B80	14	1049		4.0	
B81	14	1118		4.0	
B82	14	1136		4.0	
B83	14	1240		4.0	
B84	14	1307		5.0	
B85	14	1331		4.0	
B86	14	1401		4.0	
B87	14	1431		4.0	
B88	14	1454		4.0	
B89	14	1517		4.0	
B90	14	1540		4.0	
B91	15	0923		4.0	
B92	15	0943		4.0	Gates 5-11 closed
B93	15	1007		4.0	
B94	15	1034		4.0	
B95	15	1101		4.0	
B96	15	1123		4.0	
B97	15	1221		4.0	
B98	15	1247		5.0	
B99	15	1310		4.0	
B100	15	1326		4.0	
B101	15	1350		4.0	
B102	15	1422		4.0	
B103	15	1522		4.0	
B104	15	1542		4.0	
B105	15	1603		4.0	
B106	15	1626		4.0	
B107	16	0835		4.0	
B108	16	0900		5.0	
B109	16	0922		4.0	
B110	16	1011		4.0	
B111	16	1036		6.0	
B112	16	1057		4.0	
B113	16	1130		4.0	
B114	16	1308		5.0	
B115	16	1340		7.0	
B116	16	1407		5.0	
B117	16	1453		8.0	
B118	17	1403		5.0	
B119	17	1423		5.0	No detonation
B120	17	1428		5.0	
B121	17	1450		5.0	
B122	17	1509		5.0	
B123	17	1528		5.0	
B124	17	1600 (est)		5.0	Final charge

Table 3
Old River Gate Loads
Bay 5 - Crane

Test No.	Date 1975	Diff. Head ft	U.S. Stage Gage	North Staff Gage	Gate Leaves	Gate Opening ft	Total Load, kips	Dry	Net*	Length of Test hr:min
18	May 24	6.7	53.2	31,41		43.4	184.0	166.3	+17.7	14:42
21	25	6.4	52.6	31,41		43.4	177.5	166.3	+11.2	00:01
23	25	6.4	52.6	31,41		43.4	180.5	166.3	+14.2	15:15
43	28	6.1	50.5	21,31,41		28.8	252.5	241.1	+11.4	00:33
45	28	6.1	50.5				250.5		+ 9.4	01:32
47	29	6.0	50.3				250.0		+ 8.9	00:58
49	29	6.0	50.2				252.5		+11.4	00:56
51	29	5.9	50.1				250.5		+ 9.4	00:56
53	29	5.9	50.1				251.0		+ 9.9	00:22
55	29	6.0	50.0				259.0		+17.9	00:54
58	29	6.1	49.9				251.0		+ 9.9	01:54
60	30	6.1	49.9				249.0		+ 7.9	01:46
62	30	6.2	49.9				252.0		+10.9	01:56
68	30	7.6	49.9			24.8	258.9		+17.8	00:49
72	30	7.7	49.9				254.0		+12.9	00:51
74	30	7.7	49.8				256.4		+15.3	00:52
76	31	7.8	49.7				257.1		+16.0	01:52
78	31	7.8	49.6				254.7		+13.6	02:06
87	31	7.8	49.3				247.6		+ 6.5	01:05
93	31	7.7	49.2				251.9		+10.8	00:25
97	31	7.7	49.1				251.2		+10.1	00:82
99	31	7.7	49.1				251.9		+10.8	00:58
101	Jun 1	7.6	49.0				250.4		+ 9.3	00:52
103	1	7.6	49.0				248.7		+ 7.6	01:58
117	1	7.3	48.4				247.5		+ 6.4	01:05
119	1	7.3	48.4				249.5		+ 8.4	00:48
123	1 & 2	7.3	48.3				250.8		+ 9.7	00:49
125	2	7.3	48.1				253.4		+12.3	01:52
138	2	7.0	47.5				249.5		+ 8.4	02:02
140	2	6.9	47.5				251.7		+10.6	00:40
144	2	6.9	47.4				252.8		+11.7	01:55
146	3	6.8	47.3				250.3		+ 9.2	01:53
148	3	6.6	47.1				249.2		+ 8.1	02:00
160	3	6.2	46.7				257.8		+16.7	01:02
162	3	6.1	46.6				252.3		+11.2	00:53
166	3 & 4	6.0	46.5				246.9		+ 5.8	01:00
168	4	6.0	46.4				248.8		+ 7.7	01:53
170	4	5.9	46.2				246.4		+ 5.3	02:23
180	4	5.7	45.6				249.9		+ 8.8	00:39
184	4	5.6	45.5				247.7		+ 6.6	00:52
186	4 & 5	5.5	45.4				248.6		+ 7.5	01:54
188	5	5.5	45.3				240.2		- 0.9	01:52

* + = Downpull; - = Uplift.

(Continued)

Table 3 (Continued)

Test No.	Date 1975	Diff. ft	U.S. Stage		Gate Opening ft	Load, kips			Length of Test hr:min
			North Staff Gage	Gate Leaves		Total	Dry	Net*	
202	Jun 5	5.3	45.0	2L, 3L, 4L	24.8	246.0	241.1	+ 4.9	02:17
204		5	5.3	44.9		251.1		+10.0	00:45
206		5	5.3	44.8		248.7		+ 7.6	00:48
208		6	5.3	44.7		248.2		+ 7.1	01:52
210		6	5.2	44.5		248.5		+ 7.4	02:41
219		6	5.0	44.1		243.6		+ 2.5	01:03
223		6	5.0	44.0		245.0		+ 3.9	01:55
225	6 & 7	4.9	43.9			243.0		+ 1.9	01:52
227		7	4.8	43.7		242.9		+ 1.8	01:51
240		7	4.6	43.3		248.2		+ 7.1	00:46
242		7	4.6	43.2		250.4		+ 9.3	00:55
244		7	4.6	43.2		248.2		+ 7.1	01:54
246		8	4.6	43.1		254.8		+13.7	01:56
258		8	4.8	43.5		249.6		+ 8.5	02:04
262		8	4.9	43.5		250.7		+ 9.6	01:53
264	9	4.8	43.4			248.1		+ 7.0	06:01
276	9 & 10	4.5	43.1			249.0		+ 7.9	08:31

* + = Downpull; - = Uplift.

Table 4
Old River Gate Loads
Bay 6, Crane

Test No.	Date 1975	Diff. Head ft	U.S. Stage Gage	Gate		Load, kips			Length of Test hr:min
				North Staff Gage	Gate Leaves	Opening ft	Total	Dry	
6	May 22	7.2	54.1	6-31	7-41	43.4	176.6	166.3	+10.3 14:12
14	24	6.7	53.3				177.0		+10.7 00:01
16	24	6.7	53.2				173.0		+ 6.7 00:03
20	25	6.5	52.8				179.8		+13.5 00:10
24	26	6.2	52.3				176.6		+10.3 00:01
26	26	6.2	52.2				174.0		+ 7.7 00:13
28	26	6.1	52.0				172.0		+ 5.7 00:01

* + = Downpull; - = Uplift.

Table 5
Old River Gate Loads
Bay 7 - Crane

Test No.	Date 1975	Dift. Head ft	U.S. Stage North Staff Gage	Gate Leaves	Gate Opening ft	Load, kips			Length of Test hr:min
						Total	Dry	Ret.	
9	May 23	7.0	54.0	3L,4L	45.4	169.0	166.3	+ 2.7	13:48
41	23	6.0	50.7	2L,3L,4L	48.8	236.5	241.1	- 4.6	00:31
44	23	6.1	50.5			238.0		- 3.1	00:27
46	23	6.0	50.4			240.5		- 0.6	01:40
48	23	6.0	50.3			237.0		- 4.0	00:58
50	29	5.9	50.2			237.0		- 4.0	01:09
52	29	5.9	50.1			231.5		- 9.6	00:53
57	29	6.0	49.9			240.0		- 1.0	01:00
61	30	6.0	49.9			238.5		- 2.6	02:01
71	30	7.7	49.0		48.8	241.3		+ 0.2	00:59
73	30	7.8	49.9			246.9		+ 5.8	00:59
75	30 & 31	7.7	49.8			243.8		+ 2.7	01:55
77	31	2.8	49.7			240.6		- 0.5	01:53
88	31	7.7	49.2			240.6		- 0.5	00:54
96	31	7.7	49.2			241.3		+ 0.2	00:48
98	31	7.7	49.1			237.1		- 4.0	01:50
100	Jun 1	7.6	49.0			237.1		- 4.0	00:48
100	1	7.6	49.0			234.9		- 6.2	00:55
104	1	7.5	48.9			231.4		- 9.7	00:10
114	1	7.3	48.5			238.0		- 3.1	01:53
113	1	7.3	48.4			238.0		- 3.1	00:54
122	1	7.3	48.3			236.4		- 4.7	01:00
124	2	7.3	48.2			234.9		- 6.2	01:56
126	2	7.3	48.0			235.1		- 6.0	00:58
135	2	7.0	47.6			236.1		- 5.0	01:04
139	2	6.9	47.5			241.3		+ 0.2	00:52
143	2	6.9	47.4			239.2		- 1.9	00:55
145	2 & 3	6.8	47.3			240.0		- 1.1	13:54
147	3	6.7	47.1			233.9		- 7.2	01:47
157	3	6.3	46.8			239.3		- 1.8	00:01
161	3	6.2	46.7			238.1		- 3.0	00:52
163	3	6.1	46.6			235.7		- 5.4	00:50
167	4	6.0	46.5			238.0		- 3.1	00:51
169	4	5.9	46.3			232.7		- 8.4	01:52
172	4	5.8	45.7			239.3		- 1.8	01:06
183	4	5.7	45.6			239.3		- 1.8	00:56
185	4	5.6	45.4			234.9		- 6.2	00:56
187	5	5.5	45.3			234.9		- 6.2	01:54
189	5	5.5	45.3			234.1		- 7.0	01:00
203	5	5.3	44.9			235.1		- 6.0	00:54
205	5	5.3	44.8			230.5		- 10.6	01:02
207	5 & 6	5.2	44.8			235.9		- 5.2	01:56
209	6	5.3	44.6			231.9		- 9.0	01:57
222	6	5.0	44.1			228.8		- 12.3	01:51
224	6	5.0	43.2			231.0		- 10.1	01:52
226	7	4.9	43.8			225.7		- 15.4	01:51
228	7	4.8	43.6			225.8		- 15.3	00:02
241	7	4.6	43.3			232.7		- 8.4	00:52
243	7	4.6	43.2			234.9		- 6.2	01:56
245	8	4.6	43.1			230.5		- 10.6	01:51
247	8	4.5	43.0			234.9		- 6.2	02:06
257	8	4.7	43.3			230.9		- 8.2	00:59
261	8	4.9	43.5			236.2		- 4.0	01:11
263	8 & 9	4.8	43.4			235.5		- 5.6	01:54
273	9	4.5	43.1			238.5		- 8.6	00:16

* - Downpull; + - Uplift.

Table 6
Old River Gate Loads

Table 7
Old River Gate Loads
May 6 - Dogging Devices

Test No.	Date 1975	U.S. Stage	Gate	Dry Gate Weight						Loads, kips						Length of Tests hr:min	
				Head ft	North Staff	Gate Leaves	Gate kips	Gate ft	Gate Opening ft	North		South		North			
										Mean	Total	Mean	Total	Mean	Total		
7	May 22	7.1	54.0	6-3L,7-4L	166.3	43.4	96.0	88.3	184.3	18.0	91.6	85.6	177.2	+ 10.9	07:18		
8	22	7.1	54.0				120.0	113.3	233.3	67.0	90.6	89.1	179.7	+ 13.4	15:05		
13	23	6.9	53.6				108.2	121.8	230.0	63.7	83.4	89.9	173.3	+ 7.0	14:11		
13A	23	6.7	53.3				88.5	88.0	176.5	10.2	87.6	86.8	174.4	+ 8.1	04:40		
15	24	6.7	53.2				91.2	85.3	176.5	10.2	87.0	80.7	167.7	+ 1.4	03:39		
17	24	6.7	53.2				108.8	97.9	206.7	40.4	86.0	76.6	162.6	- 3.7	15:11		
19	25	6.5	52.8				94.4	89.4	183.8	17.5	89.6	85.8	175.4	+ 9.1	01:05		
22	25	6.4	52.6				106.4	117.3	223.8	57.5	85.6	90.9	176.5	+ 10.2	15:29		
25	26	6.2	52.3				95.4	85.5	180.9	14.6	91.0	84.6	175.6	+ 9.3	04:27		
29	26	6.2	51.6				105.0	116.0	221.0	54.7	84.8	89.7	174.5	+ 8.2	19:28		
32	27	6.2	51.5	6-2L,6-3L,7-4L	241.1	28.8	132.0	139.0	271.0	29.9	129.4	135.6	265.0	+ 23.9	00:32		
40	28	6.2	50.7			28.8	155.0	142.9	298.9	57.8	140.0	125.0	265.0	+ 23.9	24:37		
40A	28	6.1	50.6			28.8	142.0	136.0	278.0	36.9	138.2	131.1	269.3	+ 28.2	18:01		
67	30	7.6	50.0			24.8	150.0	132.9	282.9	41.8	141.6	122.5	264.1	+ 23.0	03:55		
89	30	7.8	50.0				148.3	130.5	278.8	37.7	142.6	121.3	263.9	+ 22.8	02:45		
90	30	7.7	49.9				152.5	133.6	286.1	45.0	141.6	121.5	263.1	+ 22.0	19:15		
91	31	7.8	49.3				144.8	123.5	268.3	27.2	137.0	117.8	254.8	+ 13.7	02:10		
92	31	7.8	49.3				150.0	133.6	283.6	42.5	135.0	118.4	253.4	+ 12.3	03:10		
95	31	7.7	49.2				144.7	128.6	273.3	32.2	134.6	118.8	253.4	+ 12.3	22:09		
116	Jun 1	7.3	48.5				147.4	133.8	281.2	40.1	131.0	115.4	246.4	+ 5.3	03:12		
121	1	7.3	48.3				146.4	131.8	278.2	37.1	132.0	117.4	249.4	+ 8.3	20:27		
137	2	7.0	47.6				147.8	134.9	282.7	41.6	128.0	115.4	243.4	+ 2.3	03:08		
142	2	6.9	47.4				145.4	134.9	280.3	39.2	128.0	116.4	244.4	+ 3.3	21:18		
159	3	6.2	46.7				145.1	135.9	281.0	39.9	124.0	116.4	240.4	- 0.7	04:09		
164	3	6.0	46.5				149.0	144.5	293.5	52.4	126.0	117.0	243.0	+ 1.9	23:00		
182	4	5.7	45.6				148.8	146.0	294.8	53.7	122.4	118.4	240.8	- 0.3	19:23		
221	6	5.0	44.1				148.0	152.1	300.1	59.1	129.6	130.7	260.3	+ 19.2	24:56		
239	7	4.7	43.3				139.8	141.7	281.5	40.4	123.0	124.6	247.6	+ 6.5	25:27		
260	8	4.9	43.5				142.2	140.7	282.9	41.8	126.4	123.5	249.9	+ 8.8	23:20		
275	8	4.9	43.5				142.4	133.3	275.7	34.6	126.0	126.0	261.0	+ 19.9	14:05		

* High mean did not necessarily occur at the same time.

** Total load minus dry weight.

Table 2
Inhalation Viscincent

Vertical	Vertical				Lateral/Streamwise				Transverse			
	Acceleration	Maximum	Displacement	Acceleration	Frequency	Peak-to-Peak	Displacement	Acceleration	Frequency	Peak-to-Peak	Displacement	Acceleration
Test No.	Location	Frequency (Hz)	Peak-to-Peak (in.)	in./sec.	Hz*	in.*	in./sec.	Hz*	in.*	in./sec.	in.*	in./sec.
161	Center of top gate leaf	3.61	0.061	7.06	19	0.0024	4.49	14	0.0016	6.71	0.0026	6.36
162	North side	3.62	0.211	1.61	29	0.0038	2.29	17	0.0017	3.14	0.0034	2.87
163	South side	3.63	0.211	1.07	19	0.0036	2.33	14	0.0017	3.14	0.0034	2.87
164	North side	3.64	0.061	0.91	17	0.0026	2.01	14	0.0017	2.67	0.0034	2.41
165	South side	3.65	0.061	0.81	17	0.0026	2.59	14	0.0017	2.67	0.0034	2.41
166	North side	3.66	0.055	2.92	30	0.0057	1.51	30	0.0059	1.61	0.0059	1.61
167	South side	3.67	0.055	3.30	21	0.0058	1.37	19	0.0059	1.71	0.0059	1.71
168	North side	3.68	0.052	3.55	21	0.0054	0.99	21	0.0050	2.13	0.0057	2.33
169	South side	3.69	0.052	3.61	21	0.0056	0.45	21	0.0057	2.13	0.0057	2.13
170	North side	3.70	0.056	3.61	23	0.0037	0.45	19	0.0052	2.17	0.0052	2.17
171	South side	3.71	0.051	3.45	23	0.0037	0.62	19	0.0052	2.17	0.0052	2.17
172	North side	3.72	0.057	0.63	19	0.0021	0.62	19	0.0039	2.06	0.0039	2.06
173	South side	3.73	0.057	5.23	21	0.0026	0.49	21	0.0021	2.25	0.0021	2.25
174	North side	3.74	0.012	1.22	21	0.0026	1.17	19	0.0056	2.11	0.0056	2.11
175	South side	3.75	0.012	2.81	20	0.0015	0.37	19	0.0059	2.31	0.0059	2.31
176	North side	3.76	0.010	1.05	15	0.0016	0.58	18	0.0021	2.11	0.0021	2.11
177	South side	3.77	0.010	5.00	20	0.0019	0.39	20	0.0010	2.43	0.0010	2.43
178	North side	3.78	0.037	2.83	15	0.0019	1.73	15	0.0026	2.30	0.0026	2.30
179	South side	3.79	0.037	1.38	13	0.0013	0.63	17	0.0059	1.66	0.0059	1.66
180	North side	3.80	0.063	5.15	18	0.0016	0.49	18	0.0051	1.53	0.0051	1.53
181	South side	3.81	0.033	2.60	19	0.0020	0.45	18	0.0023	3.03	0.0023	3.03
182	North side	3.82	0.355	42.6	18	0.0190	4.93	18	0.0056	2.11	0.0056	2.11
183	South side	3.83	0.321	1.31	19	0.0017	0.96	15	0.0052	1.71	0.0052	1.71
184	North side	3.84	0.033	0.83	22	0.0022	1.35	20	0.0054	1.85	0.0054	1.85
185	South side	3.85	0.193	35.7	22	0.0376	4.65	20	0.0350	2.11	0.0350	2.11
186	North side	3.86	0.033	1.38	22	0.0029	1.50	16	0.0055	3.15	0.0055	3.15
187	South side	3.87	0.033	5.15	17	0.0023	0.65	16	0.0036	1.74	0.0036	1.74
188	North side	3.88	0.033	0.83	14	0.0075	3.24	16	0.0193	6.16	0.0193	6.16
189	South side	3.89	0.033	1.38	19	0.0253	5.83	17	0.0335	3.5	0.0335	3.5
190	North side	3.90	0.042	6.57	18	0.0024	1.61	15	0.0291	2.14	0.0291	2.14
191	South side	3.91	0.013	1.44	14	0.0021	0.87	15	0.0053	1.96	0.0053	1.96
192	North side	3.92	0.014	4.64	15	0.0012	0.43	14	0.0012	2.62	0.0012	2.62
193	South side	3.93	0.011	1.61	11	0.0012	0.51	14	0.0012	2.35	0.0012	2.35
194	North side	3.94	0.017	1.79	10	0.0015	1.22	10	0.0015	2.22	0.0015	2.22

$\frac{1}{2}$ Cycles per second.
 $\frac{1}{2}$ = acceleration due to gravity = 32.13 ft/sec^2 .

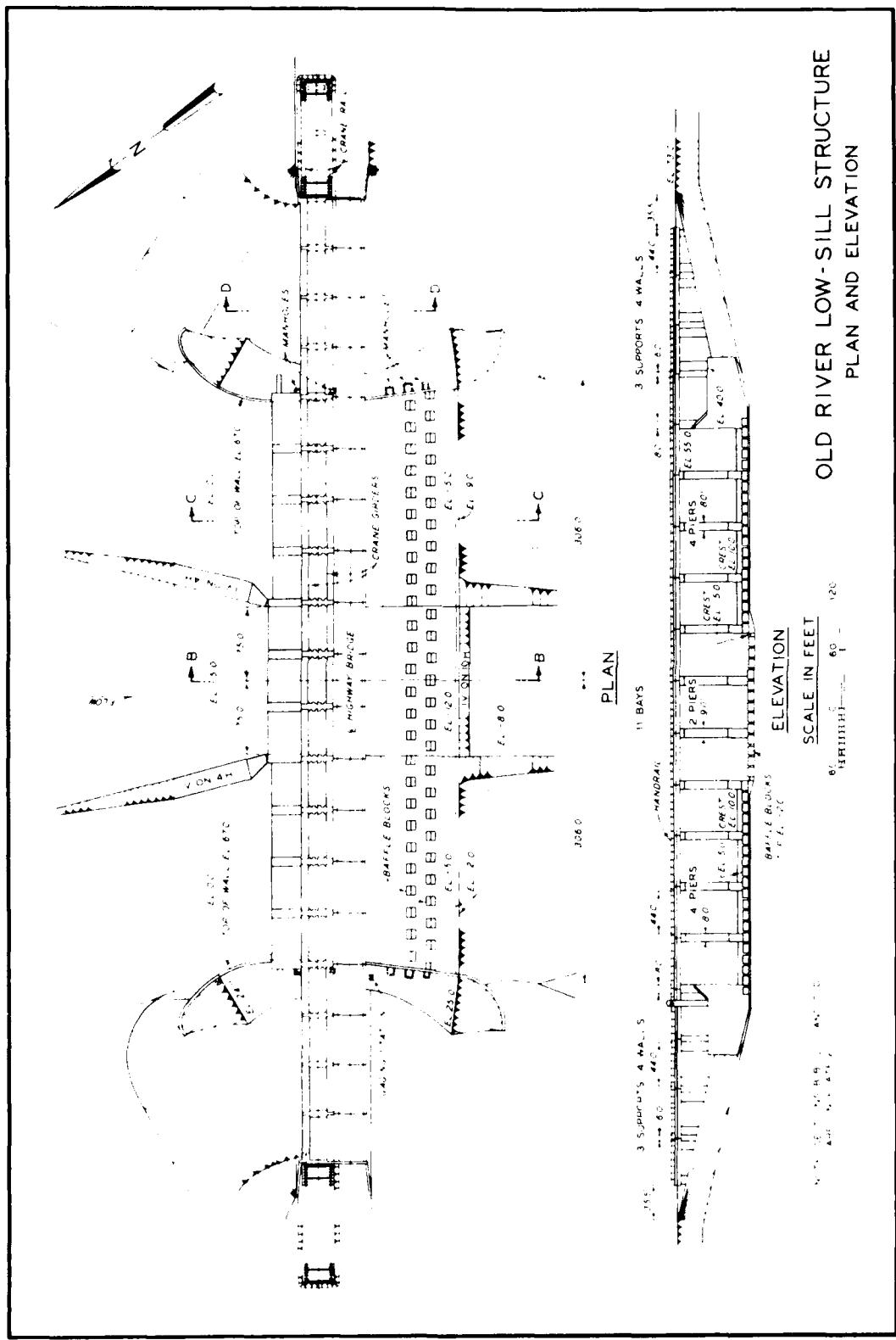
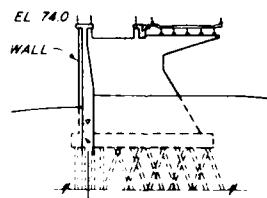
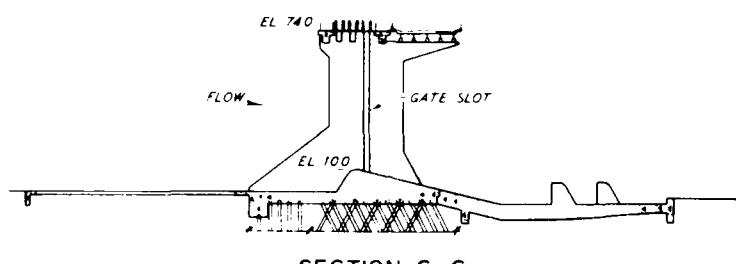
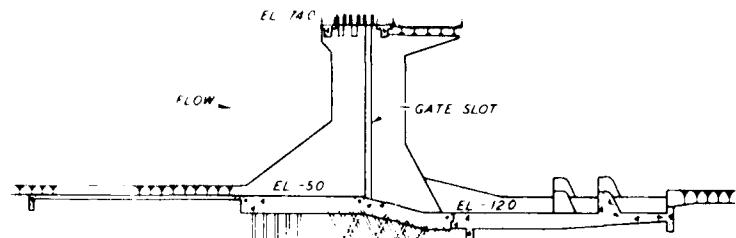


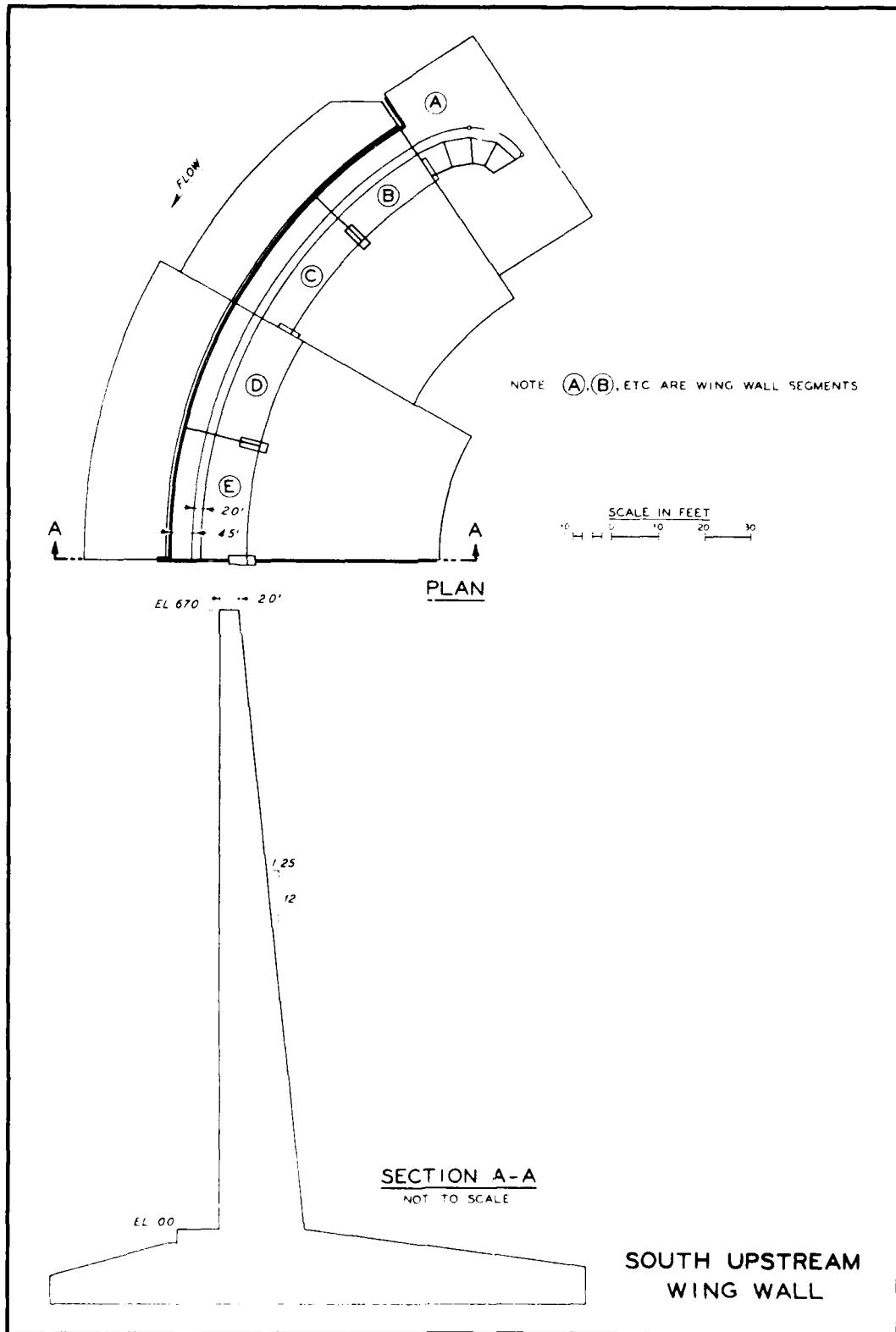
PLATE 1



SCALE IN FEET
40 0 40 80

OLD RIVER
LOW-SILL STRUCTURE
PIERS

PLATE 2



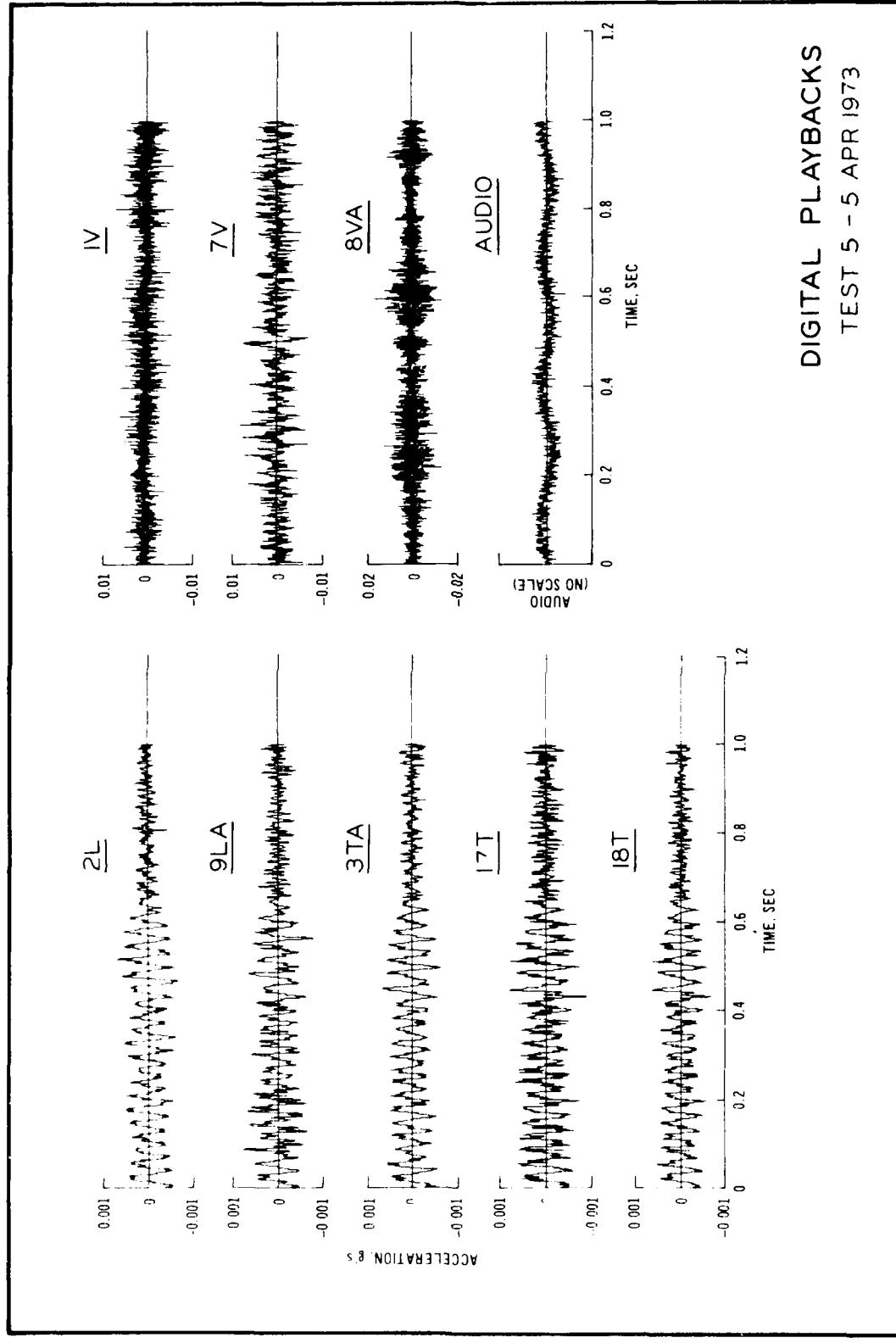


PLATE 4

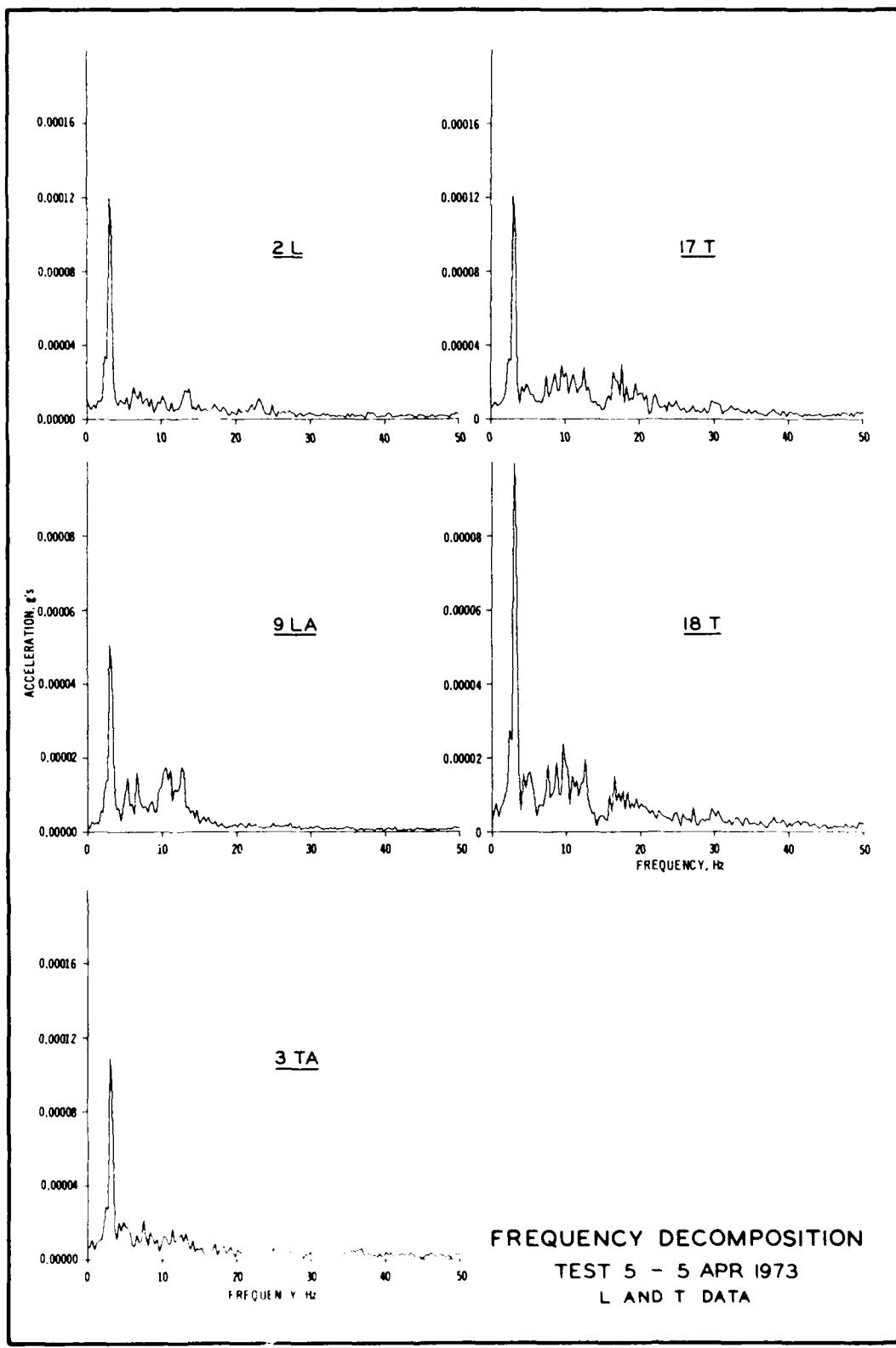
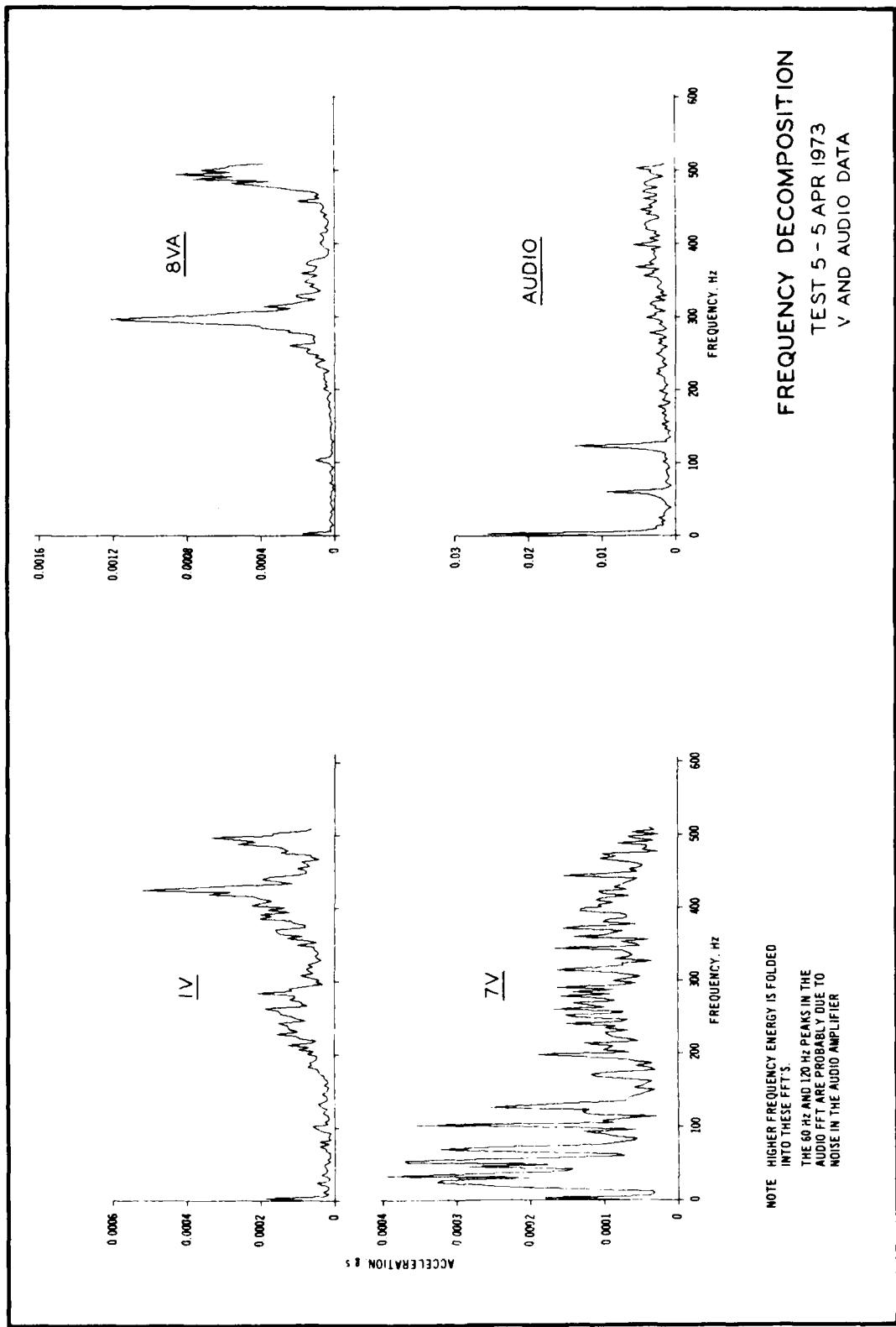
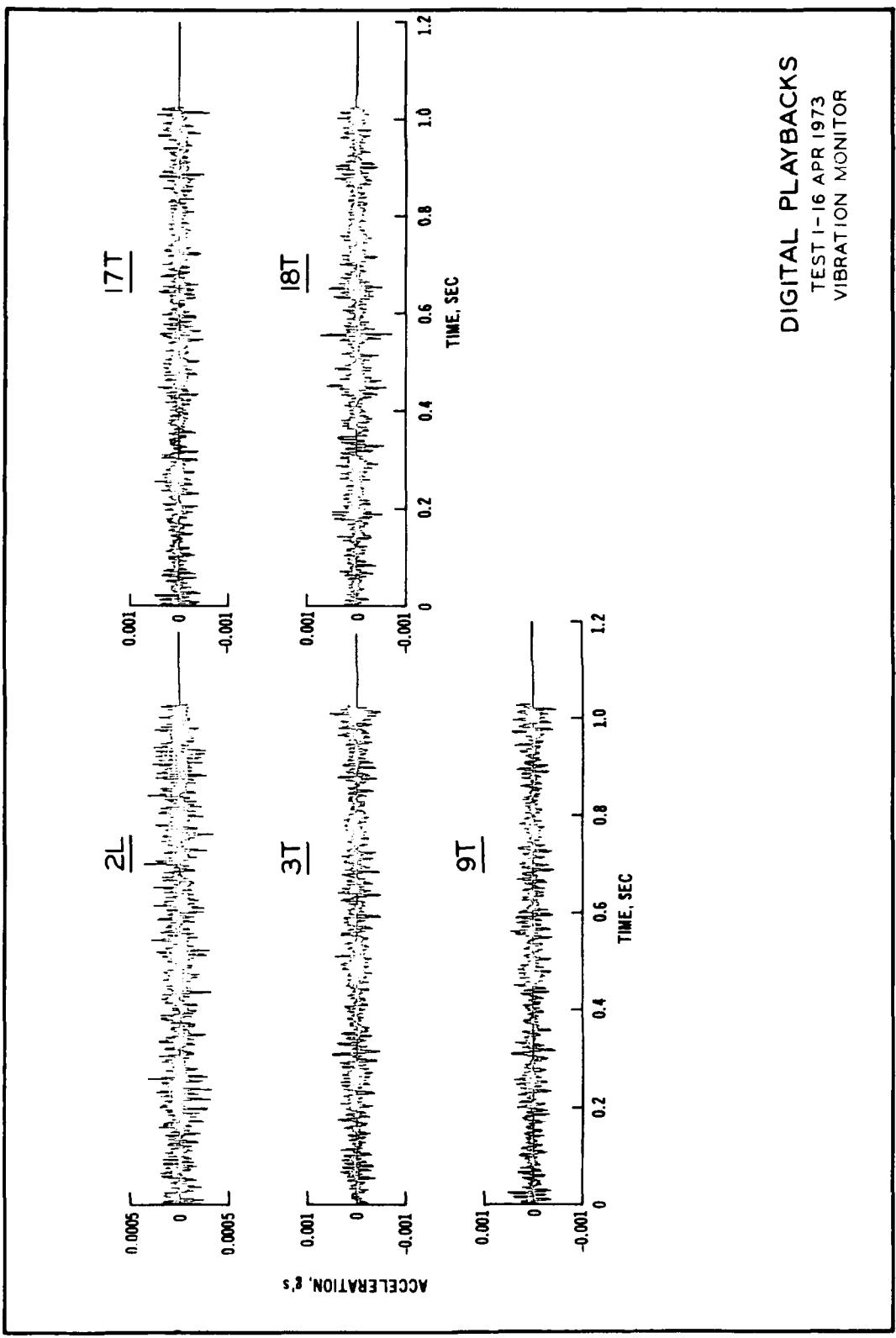


PLATE 5



FREQUENCY DECOMPOSITION
TEST 5 - 5 APR 1973
V AND AUDIO DATA

NOTE: HIGHER FREQUENCY ENERGY IS FOLDED
INTO THESE FFT'S.
THE 60 Hz AND 20 Hz PEAKS IN THE
AUDIO FFT ARE PROBABLY DUE TO
NOISE IN THE AUDIO AMPLIFIER



DIGITAL PLAYBACKS
TEST 1-16 APR 1973
VIBRATION MONITOR

PLATE 7

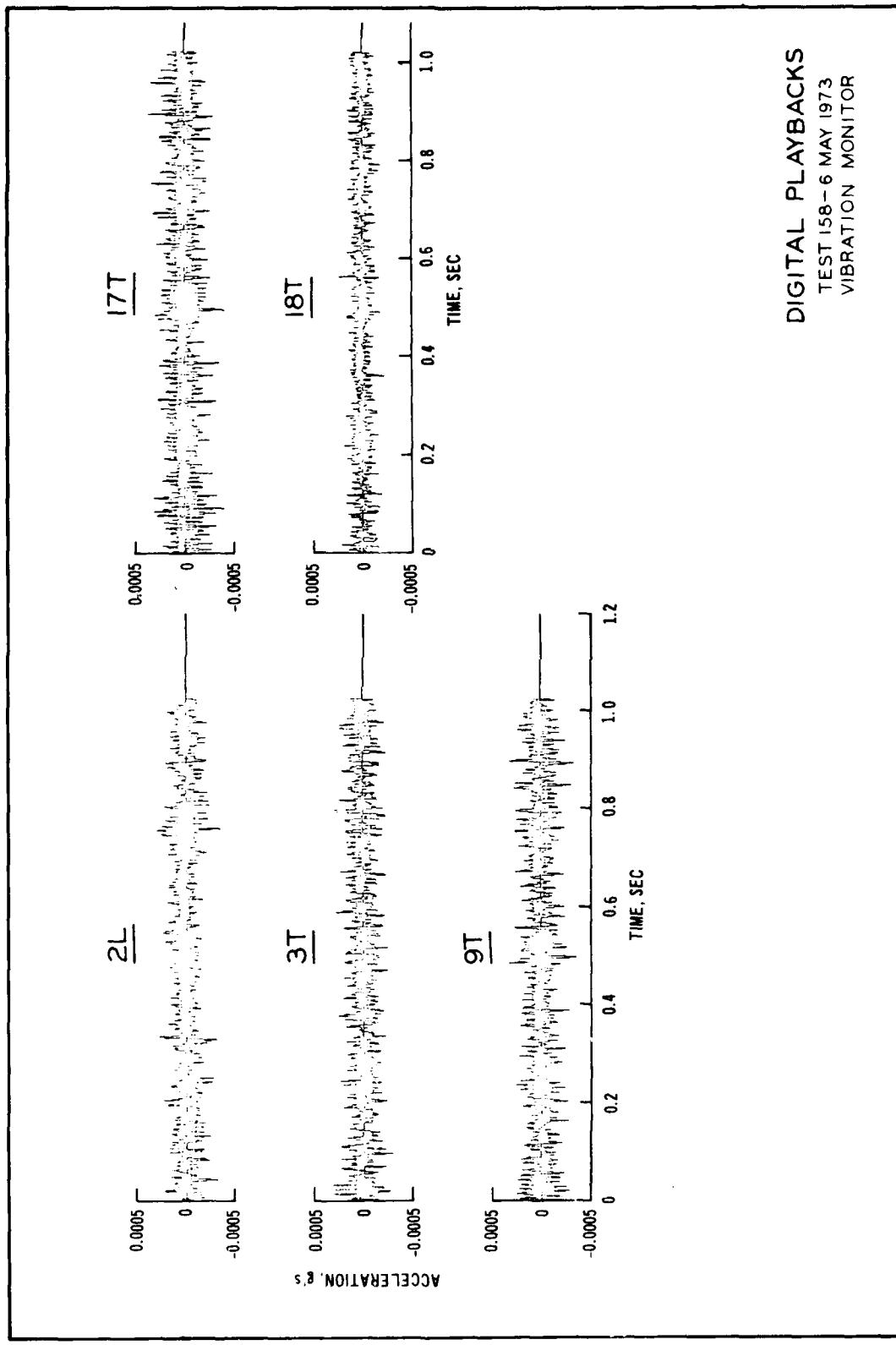


PLATE 8

DIGITAL PLAYBACKS
TEST 158-6 MAY 1973
VIBRATION MONITOR

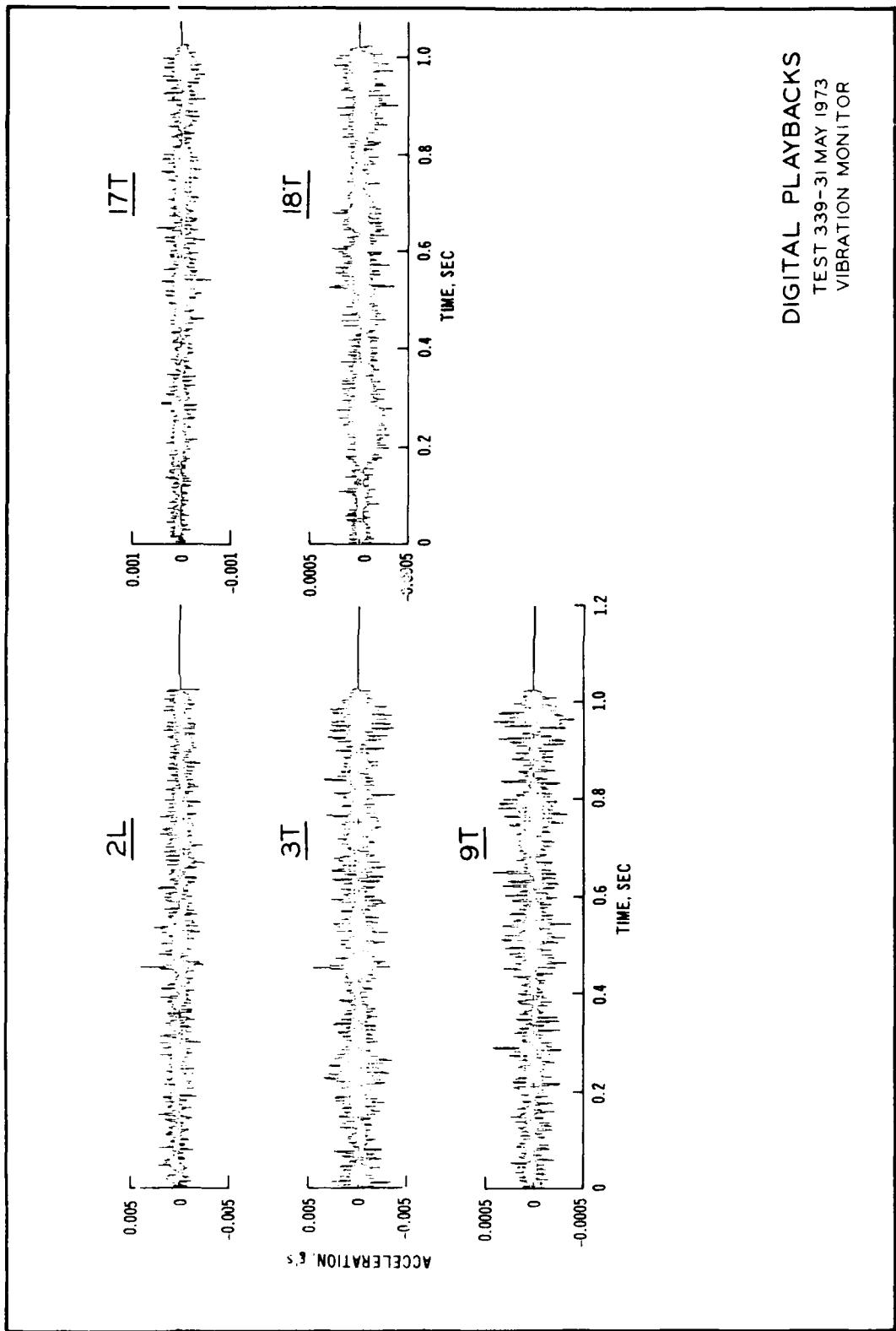


PLATE 9

DIGITAL PLAYBACKS
TEST 339-31 MAY 1973
VIBRATION MONITOR

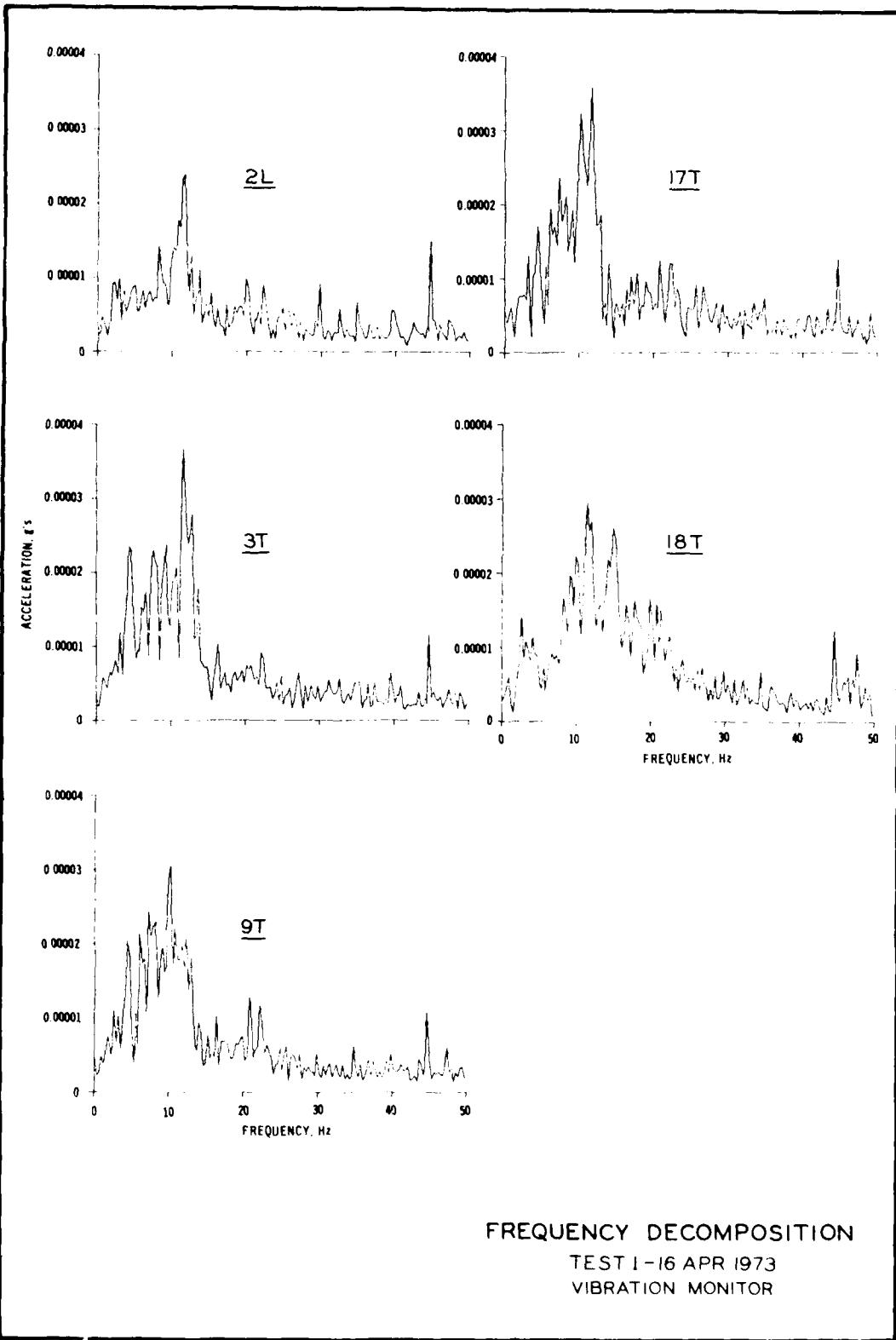
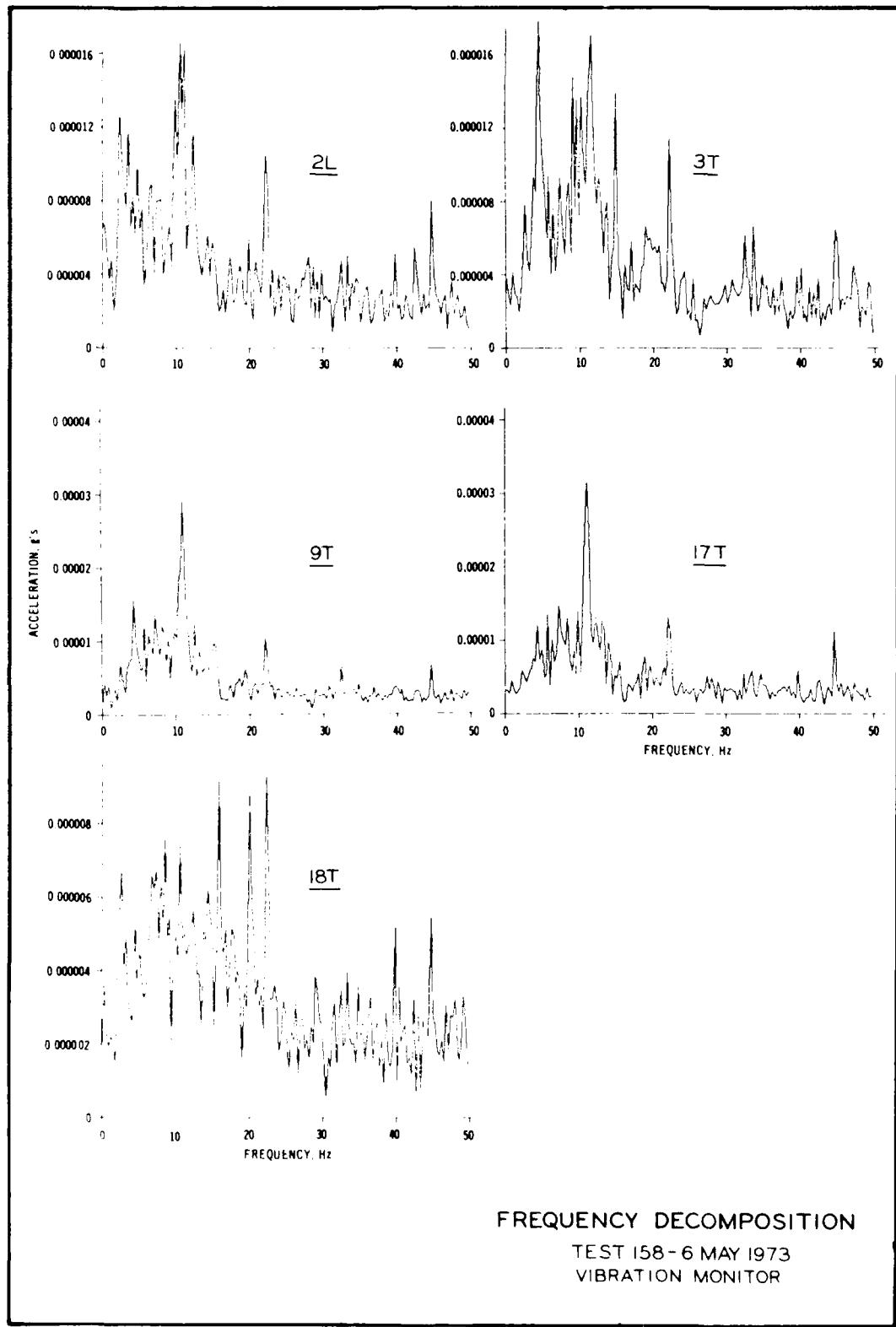


PLATE 10



FREQUENCY DECOMPOSITION

TEST 158-6 MAY 1973
VIBRATION MONITOR

PLATE 11

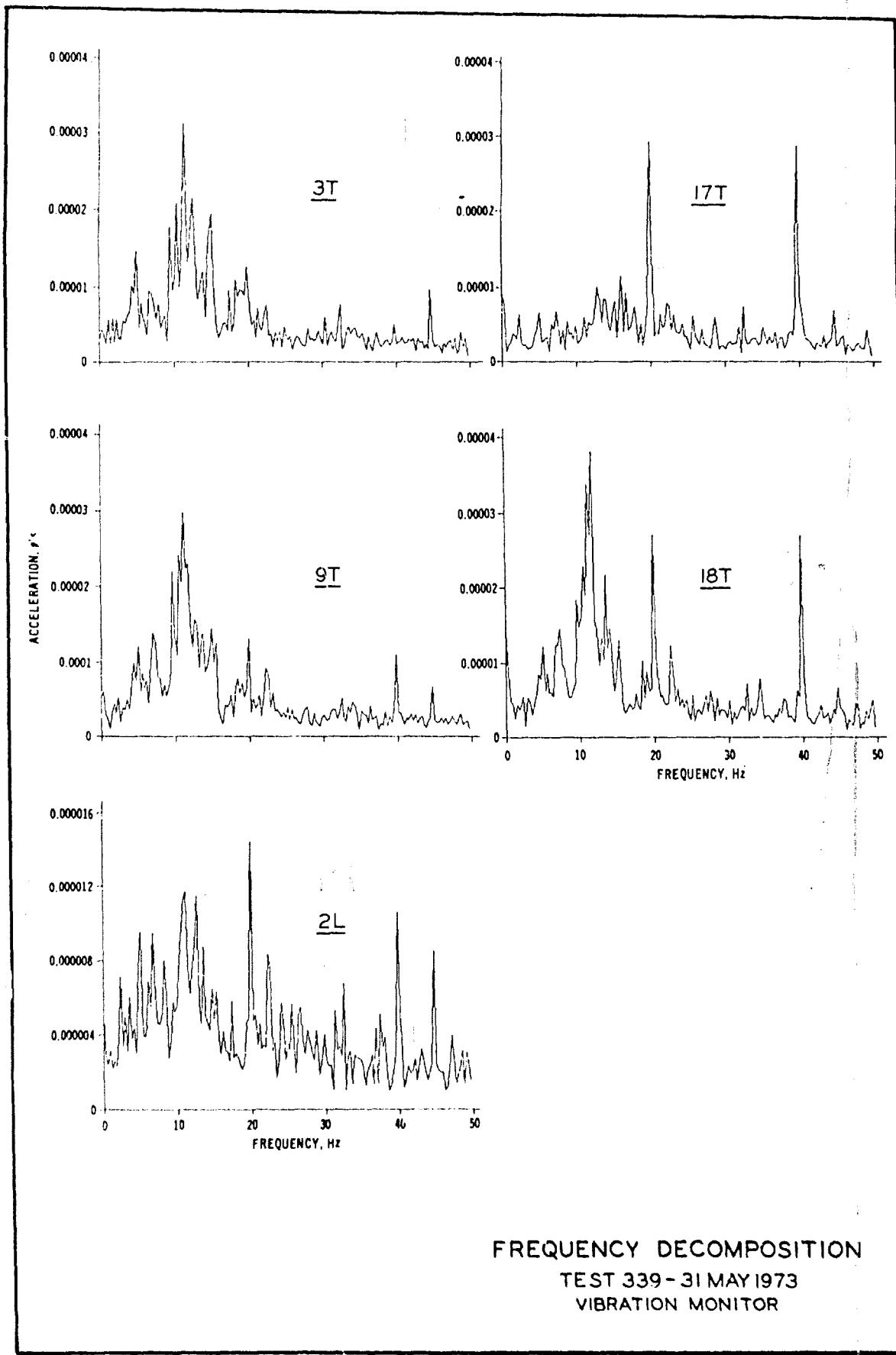
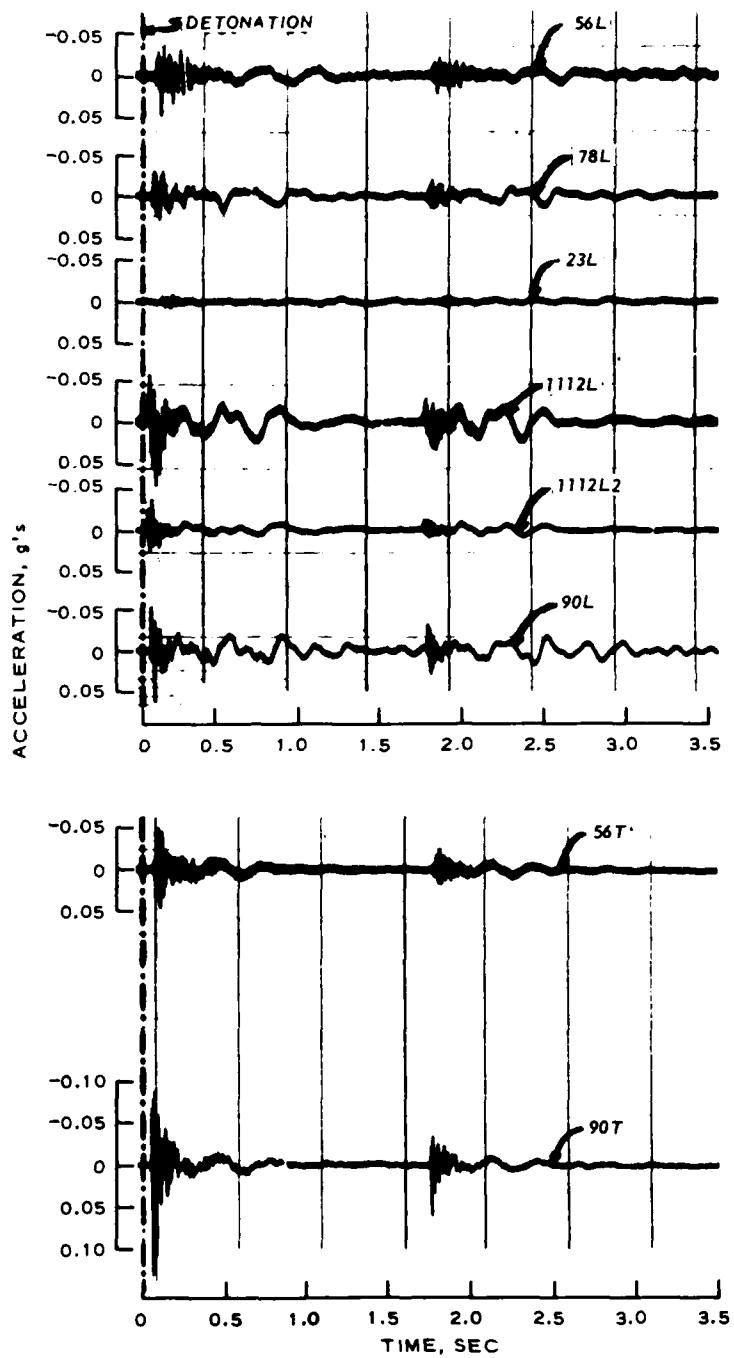


PLATE 12



ANALOG PLAYBACK
B10, LONGITUDINAL AND
TRANSVERSE ACCELERATIONS

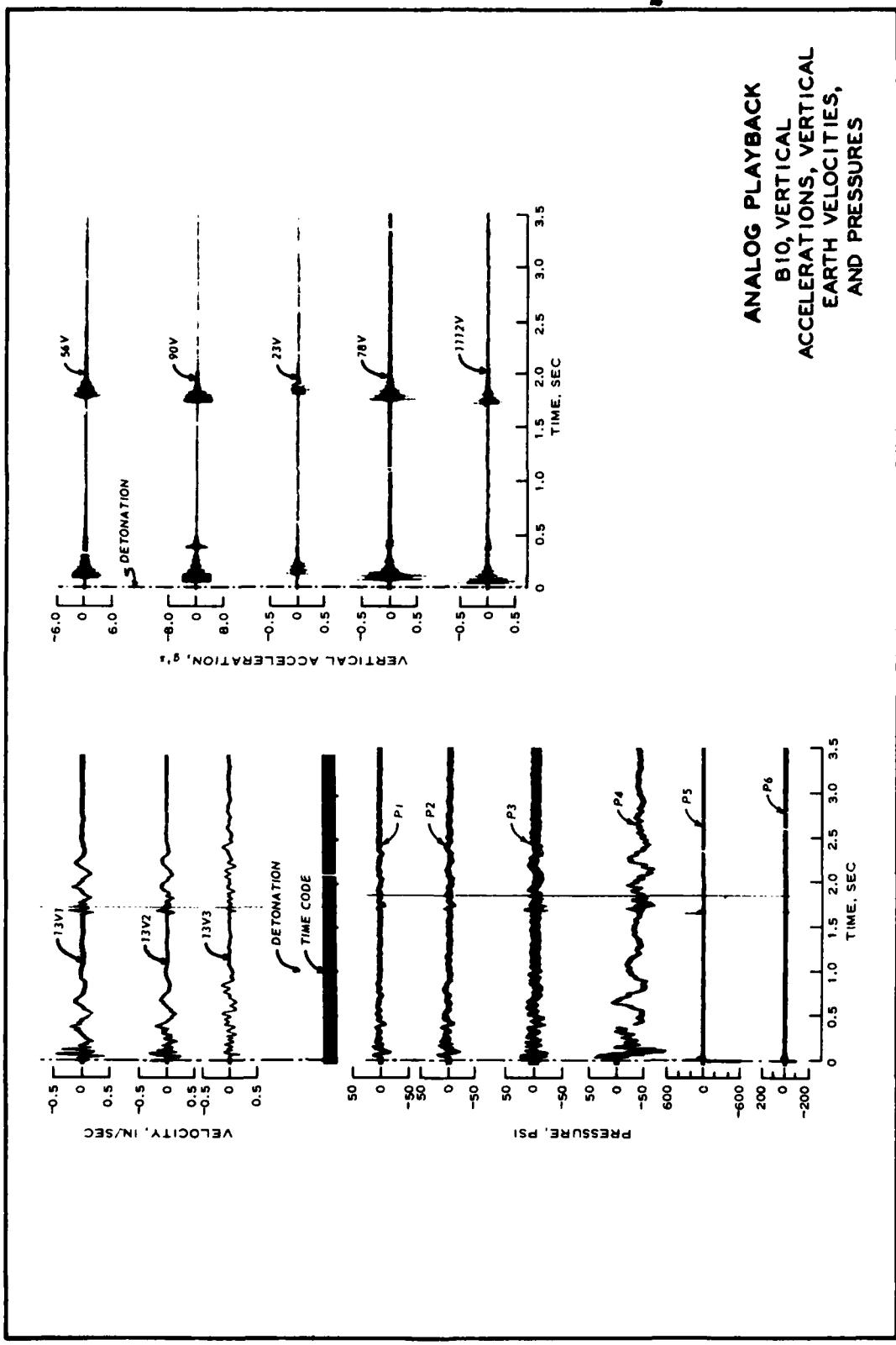


PLATE 14

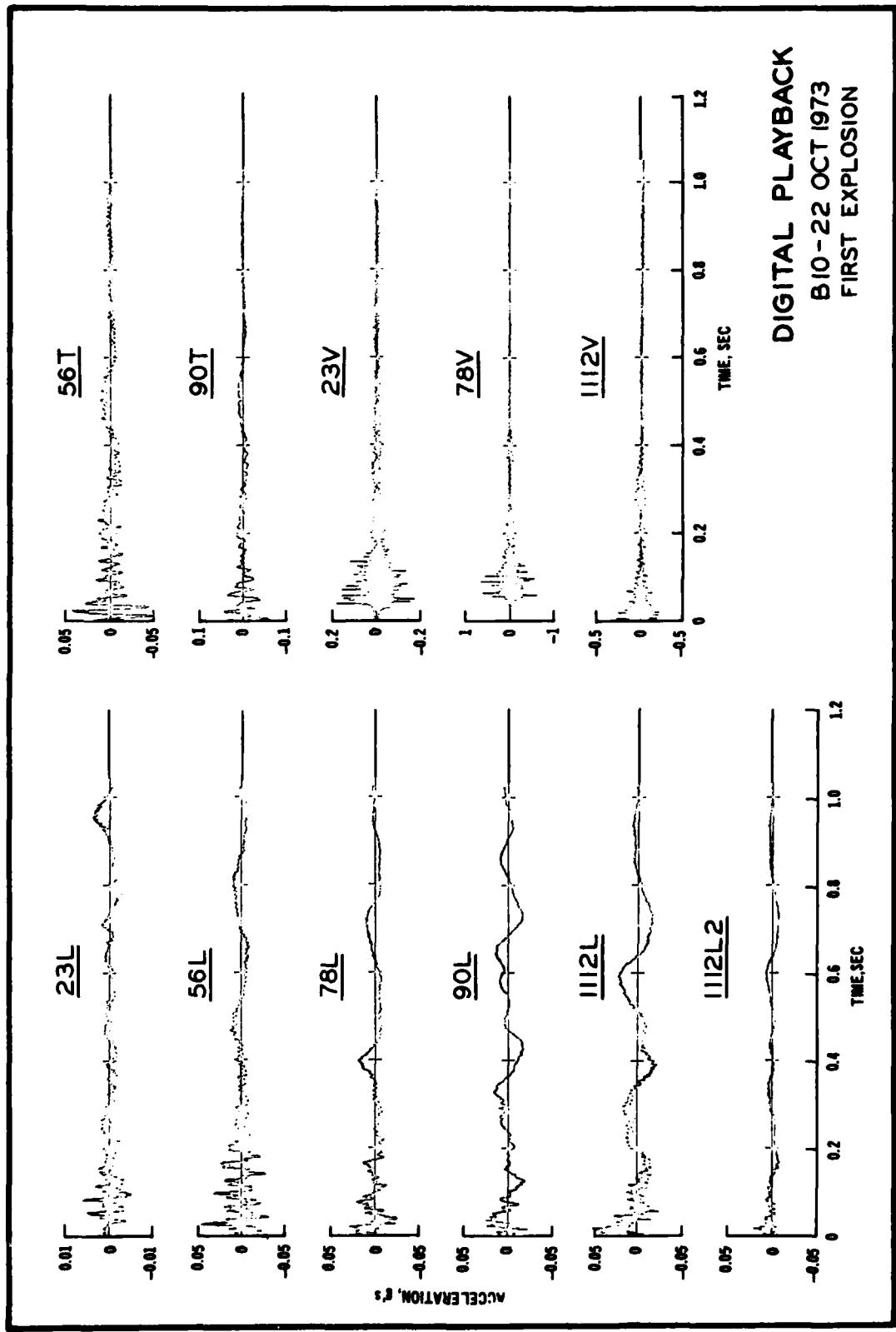


PLATE 15

DIGITAL PLAYBACK
B10-22 OCT 1973
FIRST EXPLOSION

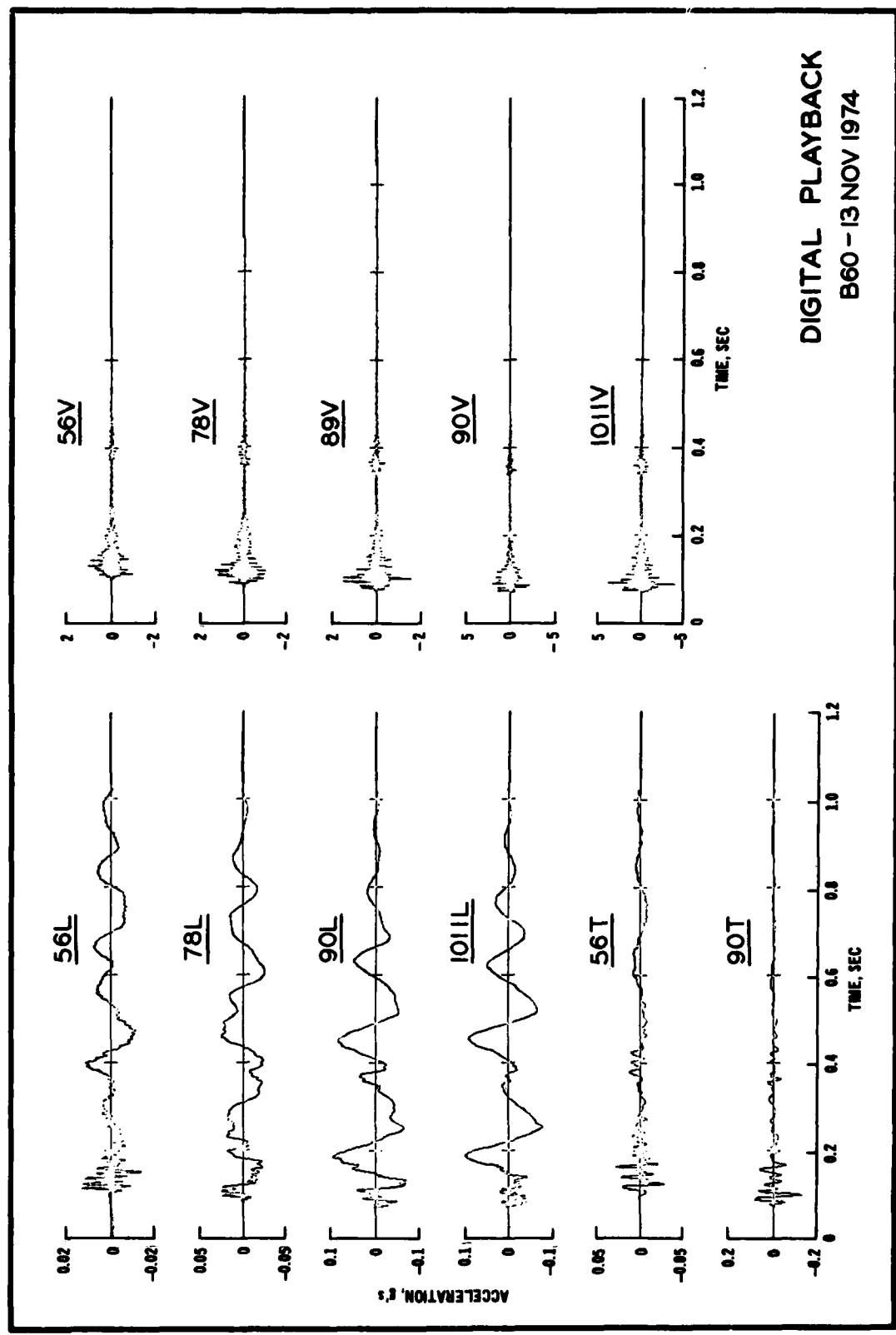
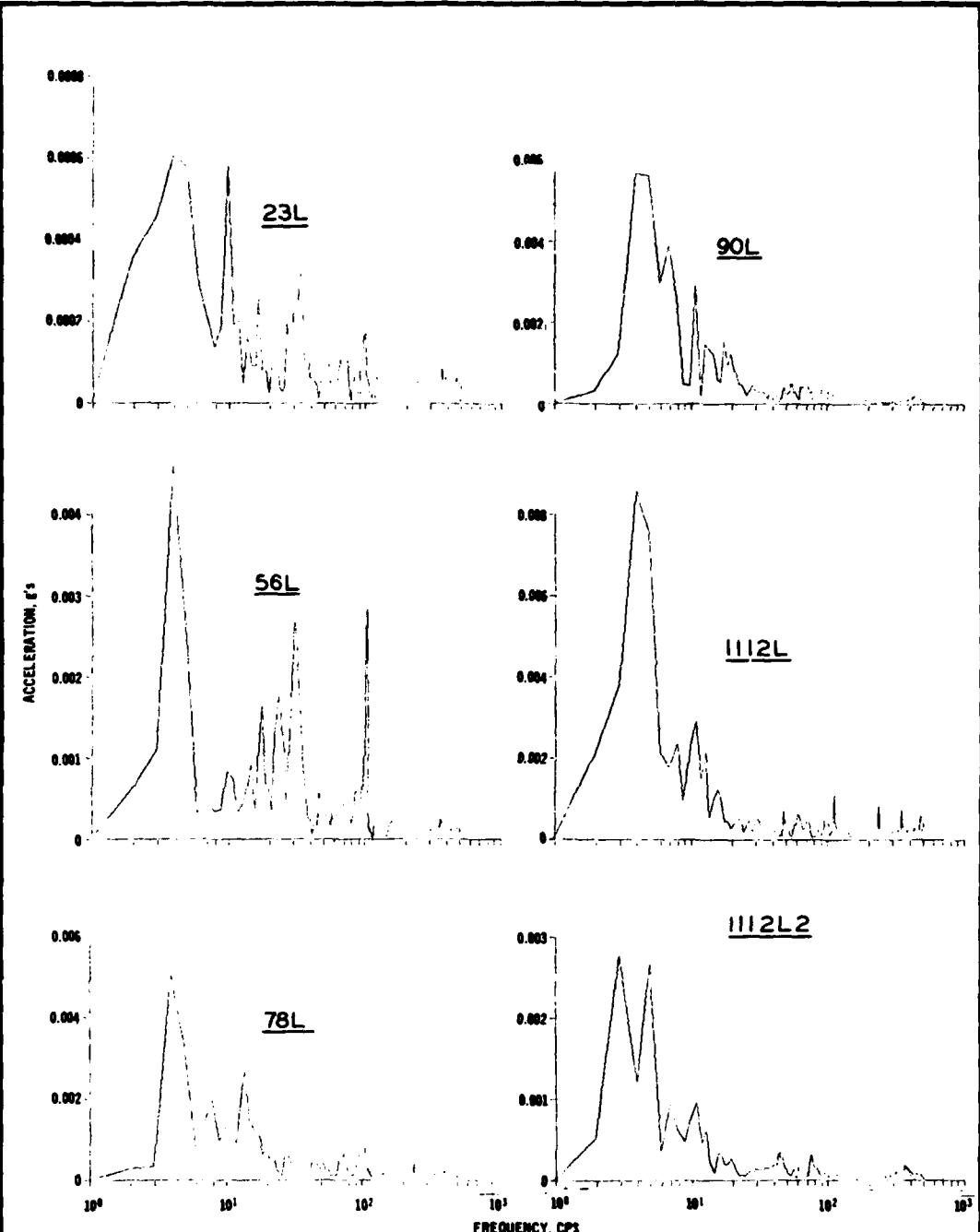


PLATE 16



FREQUENCY DECOMPOSITION
B10-22 OCT 1973
LONGITUDINAL ACCELERATIONS

PLATE 17

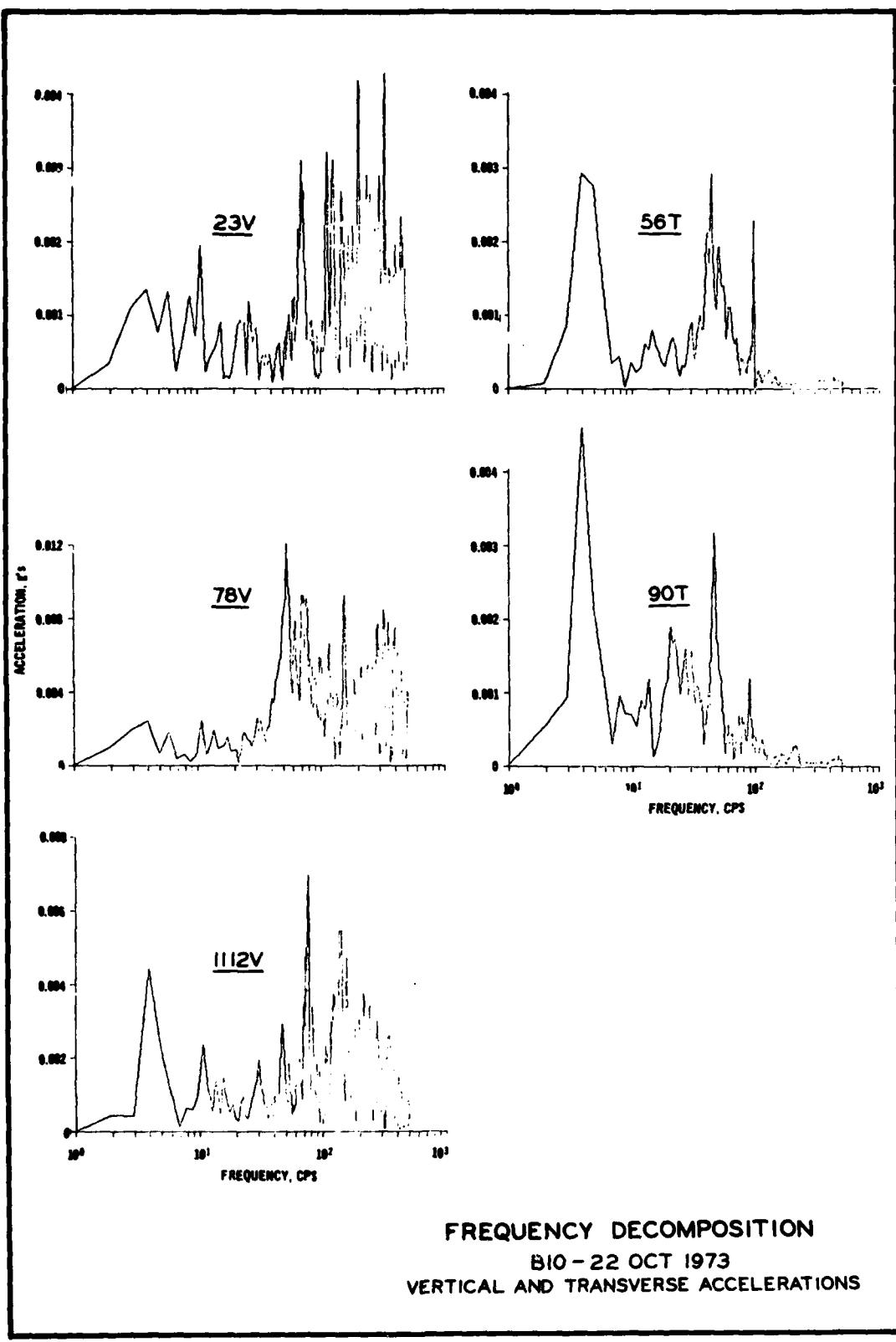
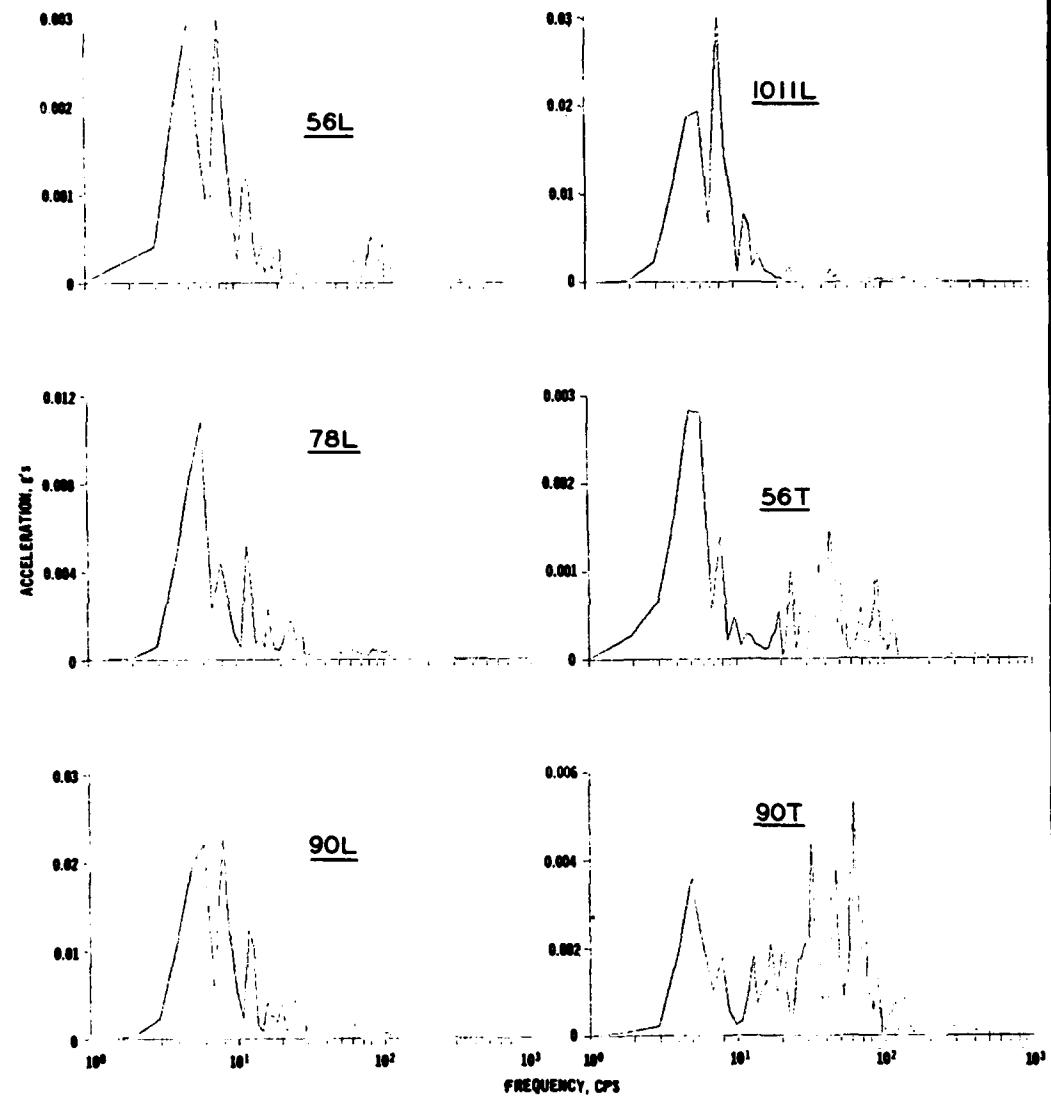
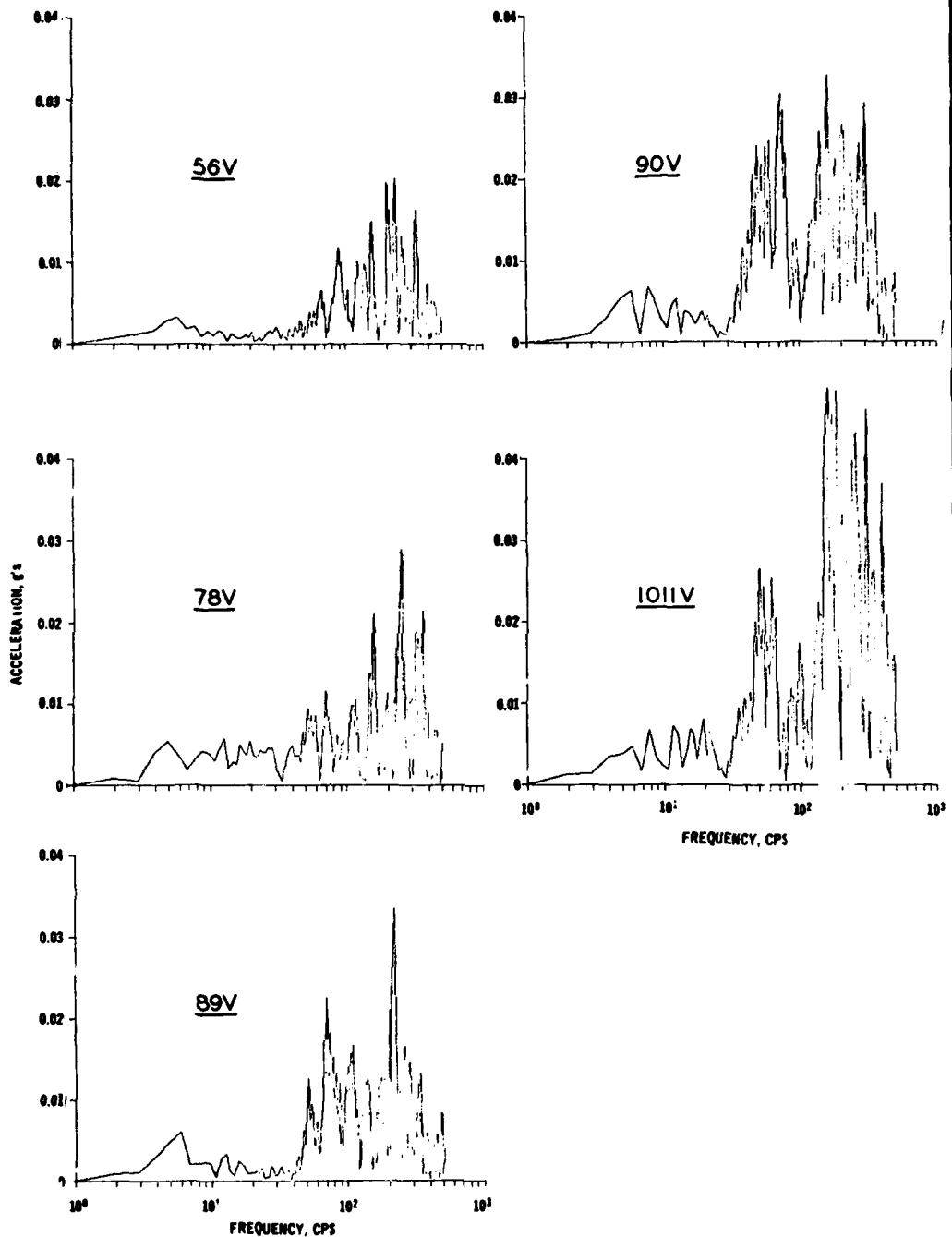


PLATE 18



FREQUENCY DECOMPOSITION
B 60 - 13 NOV 1974
LONGITUDINAL AND TRANSVERSE
ACCELERATIONS

PLATE 19



FREQUENCY DECOMPOSITION
B60-13 NOV 1974
VERTICAL ACCELERATIONS

PLATE 20

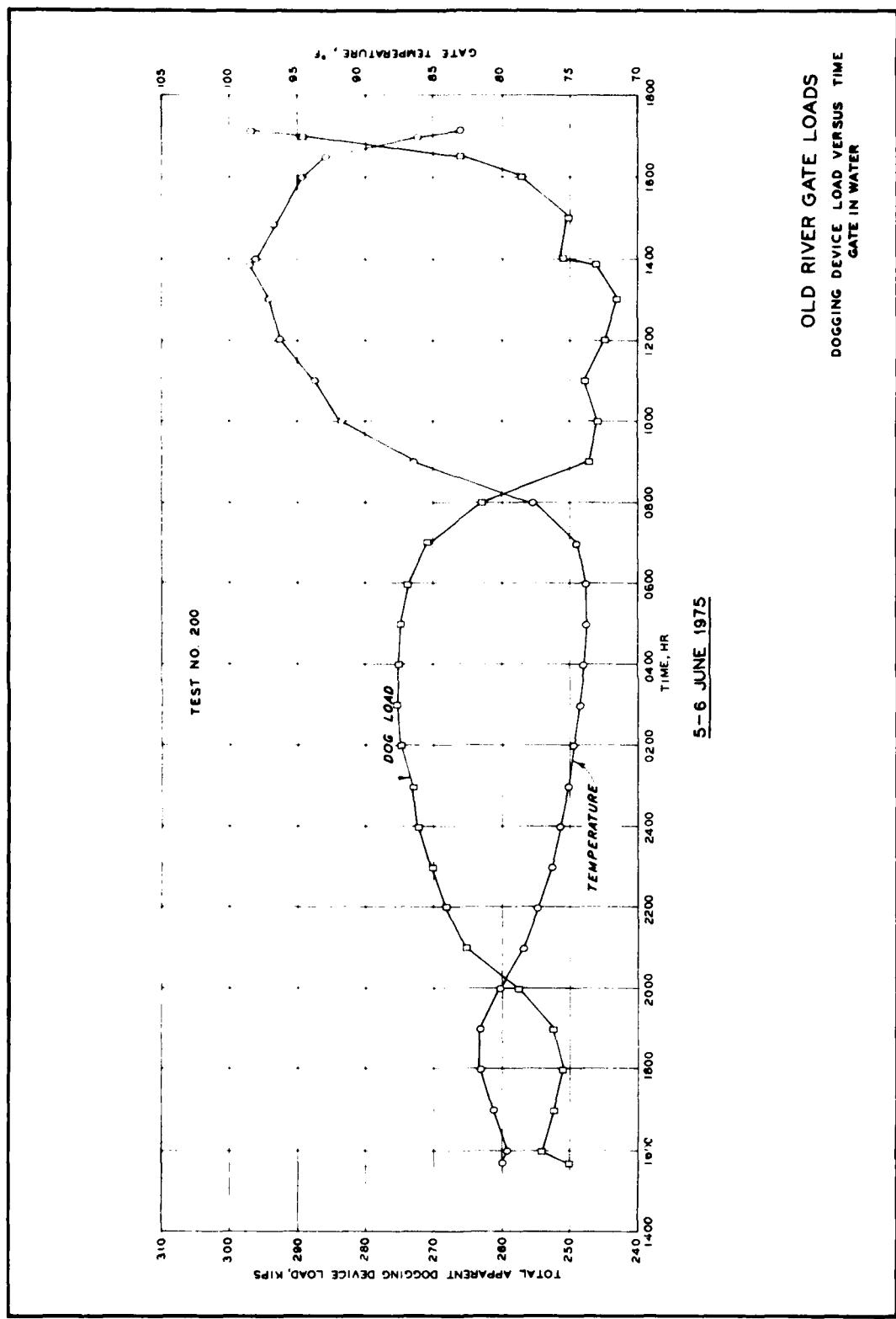
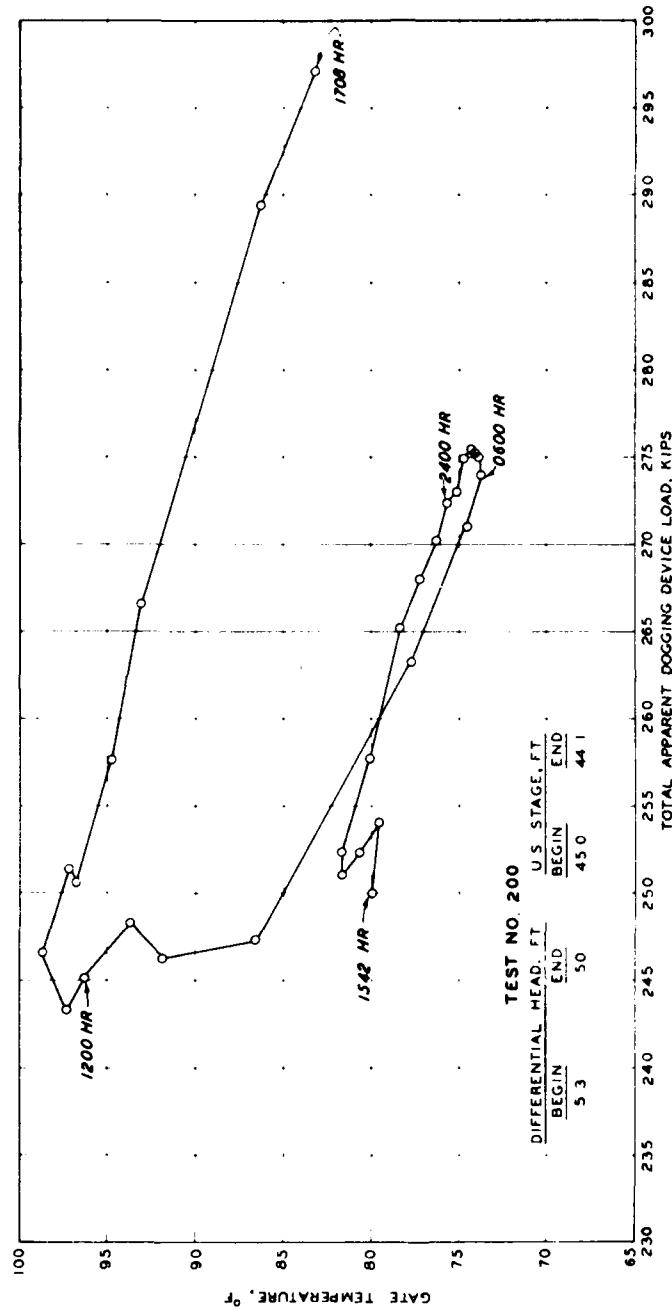


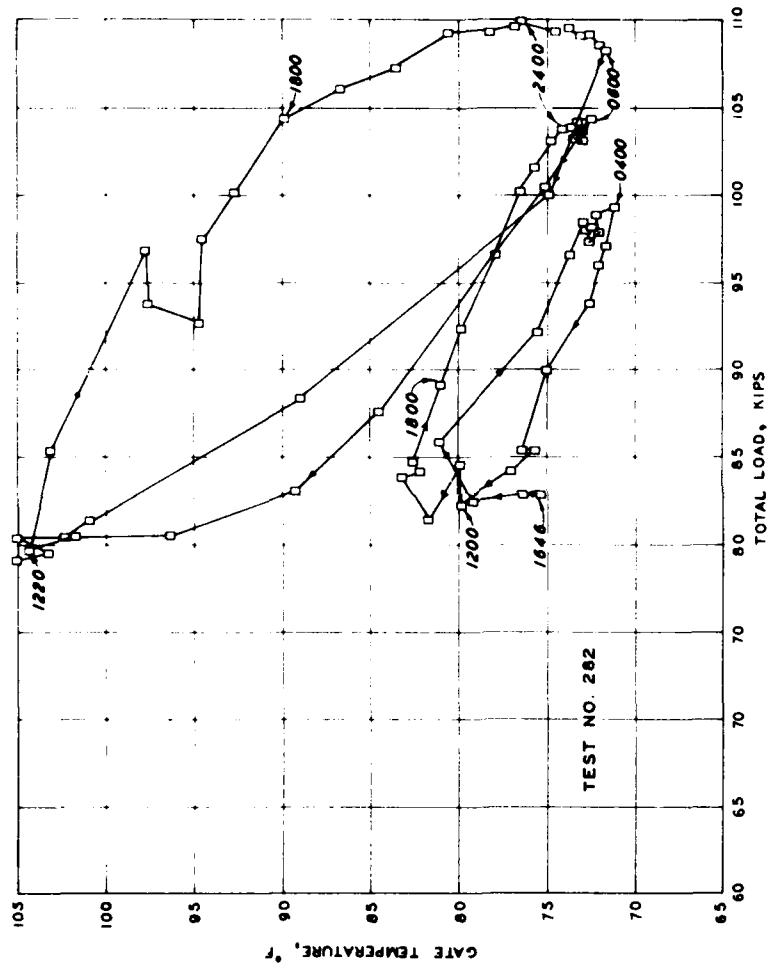
PLATE 21



OLD RIVER GATE LOADS

5-6 JUNE 1975

GATE TEMPERATURE VERSUS DOGGING DEVICE LOAD
GATE IN WATER



OLD RIVER GATE LOADS
10-13 JUNE 1975
GATE TEMPERATURE VERSUS TOTAL LOAD
GATE IN AIR

APPENDIX A: NOTATION

a^+	Acceleration (positive extreme value) LT^{-2}
a^-	Acceleration (negative extreme value) LT^{-2}
f	Frequency T^{-1}
f_n	Natural frequency T^{-1}
f_p	Dominant frequency (in FFT results) T^{-1}
f_x	Folding frequency T^{-1}
X	Displacement L
δ	Logarithmic decrement
Δt	Time (seconds) between samples
ΔV	Maximum peak-to-peak velocity change
π	3.141

In accordance with ER 70-2-3, paragraph 6c(1)(b),
dated 15 February 1973, a facsimile catalog card
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Neilson, Frank M

Prototype tests, Old River Low-Sill Control Structure,
April 1973-June 1975, by Frank M. Neilson [and] Allen R.
Tool. Vicksburg, U. S. Army Engineer Waterways Experiment
Station, 1976.

1 v. (various pagings) illus. 27 cm. (U. S. Water-
ways Experiment Station. Technical report H-76-15)

Prepared for U. S. Army Engineer District, New Orleans,
New Orleans, Louisiana.

Includes bibliography.

1. Demolition. 2. Gates (Hydraulic structures).
3. Old River Control Structure. 4. Prototype tests.
5. Vibration. I. Tool, Allen R., joint author. II. U. S.
Army Engineer District, New Orleans. (Series: U. S.
Waterways Experiment Station. Technical report H-76-15)
TA7.W34 no.H-76-15